Travel Model Speed Estimation and Post Processing Methods for Air Quality Analysis

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TMIP

Travel
Model
Improvement
Program

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Travel Model Improvements Program

The Department of Transportation, in cooperation with the Environmental Protection Agency and the Department of Energy, has embarked on a research program to respond to the requirements of the Clean Air Act Amendments of 1990 and the Intermodal Surface Transportation Efficiency Act of 1991. This program addresses the linkage of transportation to air quality, energy, economic growth, land use and the overall quality of life. The program addresses both analytic tools and the integration of these tools into the planning process to better support decision makers. The program has the following objectives:

1. To increase the ability of existing travel forecasting procedures to respond to emerging issues including; environmental concerns, growth management, and lifestyles along with traditional transportation issues,

2. To redesign the travel forecasting process to reflect changes in behavior, to respond to greater information needs placed on the forecasting process and to take advantage of changes in data collection technology, and

3. To integrate the forecasting techniques into the decision making process, providing better understanding of the effects of transportation improvements and allowing decision makers in state governments, local governments, transit operators, metropolitan planning organizations and environmental agencies the capability of making improved transportation decisions.

This program was funded through the Travel Model Improvement Program.

Further information about the Travel Model Improvement Program may be obtained by writing to:

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Final Report
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Prepared for
US Department of Transportation
 Federal Transit Administration
 Federal Highway Administration
 Office of the Secretary
US Environmental Protection Agency

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Executive Summary

Transportation planners have relatively sophisticated and complex computer models available to them for forecasting travel demand and air quality. The weak point in the process however is the interface between the demand forecasting and the pollutant emission models. Travel demand models are designed to forecast travel demand but have not traditionally been as reliable for forecasting vehicular speeds. Air pollutant models however require as input relatively reliable estimates of vehicle demand, vehicle speeds, and vehicle operating mode (eg. cold start, hot start, etc.). This gap between the traditional outputs of travel demand models and the required inputs of air quality models is the subject of this report.

This report suggests various short term improvements that might be made to the speed estimation routines contained in travel demand models, and suggests various post-processor routines that can be used to further improve model speed estimates. These post-processor routines generally use data and procedures not typically available in travel demand models. Finally this report suggests improvements that can be made in current techniques for estimating vehicle operating modes (cold start).

Purpose of Report

The purpose of this report is to present practitioners with various easily implementable strategies for improving the speed and vehicle operating mode estimates used in air quality modeling. The strategies are designed to be implemented in the short term.

Since the capabilities and resources of metropolitan planning organizations vary quite widely throughout the United States, no single set of methods is recommended. Several promising methods for improving the estimation of speed and operating mode are described for planning agencies to choose from.

Improvements to Travel Demand Models

Travel demand models typically employ one or more variations of the standard Bureau of Public Roads (BPR) speed-flow equation to estimate mean vehicle speeds. A look-up table is used to determine a street's free flow speed and capacity depending on the type of street (freeway, arterial, etc.) and the area in which the street is located (urban, suburban, rural, etc.). The BPR equation then predicts the mean vehicle speed on the street as a function of the volume/capacity ratio for the street.

The BPR equation was calibrated to data used in the 1965 Highway Capacity Manual. More recent data used in the 1994 Highway Capacity Manual however indicates that BPR equation no longer accurately represents the influence of traffic flows on vehicle speeds.

A new procedure for estimating free-flow speed, developed as part of the research project NCHRP 3-55(2), is suggested for improving the look-up table of free-flow speeds used in the BPR method. This procedure relies on the posted speed limit. Two new Highway Capacity Manual methods for estimating free-flow speeds for freeways and multi-lane rural highways are also presented.

The Florida DOT method of developing default values and entering them into the Highway Capacity Manual to estimate capacities for planning purposes is suggested for improving the look-up tables of capacities currently used in the BPR method.

Finally, two new speed-flow equations are suggested to replace the standard BPR equation. One new equation is simply an updated of the BPR equation with new parameters. The other equation was developed by Akcelik and is currently employed extensively in the Highway Capacity Manual. Node

delay computation procedures are also suggested for consideration by planners, but these specialized procedures are implementable in only two or three of the software packages currently available to planners.

Assignment Post-Processors

Assignment post-processors are suggested as one method for getting around software and data limitations that may prevent implementation of node delay estimation techniques, queuing analysis, and other refined analytical techniques at the travel demand modeling stage. Several different post-processing techniques that have been used in practice are described in detail. Each post processor technique has its strengths and weaknesses. Some are based upon simulation model analyses. Others are based on simplification of the Highway Capacity Manual techniques.

Prediction of Vehicle Operating Mode

Air pollution models require information on the number of trips and number of vehicle-miles traveled by vehicle operating mode since higher emissions of TOG and CO can be attributed to cold starts. The data required for predicting vehicles by operating mode is tied directly to the emission factor model required by the U.S. EPA. The current emission factor models account for start emissions differently. MOBILE requires the fraction of VMT in the hot and cold transient modes, while EMFAC calculates hot and cold start emissions factors that are associated with the engine start at the trip origin. The emission factors are applied to vehicular travel data derived from travel demand models. The current emission factor models, the regional models, and post-processors are discussed and current approaches are evaluated.

Short-term and long-term improvements for estimating emissions from vehicles in the start mode are recommended. Regional demand models, such as EMME2 and TRANPLAN, have been adapted to track cold start vehicles on a link-by-link basis and can be used to derive alternate operating mode fractions for input to MOBILE or alternate start mode percentages for EMFAC. Research using travel survey data has shown significant differences from the default operating mode fractions used by MOBILE5. Instrumented vehicle surveys conducted by EPA also indicate differences in travel behavior. The data from demand models, travel behavior surveys, and vehicle instrumentation studies can potentially be used provide estimates of operating mode fractions that are specific to the roadway network, time-of-day, and trip purpose. Further research is suggested to evaluate sensitivity of emission estimates to changes in operating mode fractions, to test the accuracy of the emission estimates, and to compare MOBILE5 and EMFAC results.

List of Terms

This section provides definitions for some of the more frequently used terms used in this document.

Travel Demand Model
- A set of equations and procedures for predicting travel demand as a function of economic development and the available transportation system characteristics. Travel demand models are usually created and operated within a particular computer software environment, such as TRANPLAN, MINUTP, UTPS, TMODEL, QRS, or EMME2.

Air Pollutant Emission Model
- A set of equations and procedures for predicting the air pollutant emissions produced by vehicular travel.

Post-Processor
- A set of equations and procedures for refining the speed estimates output by travel demand models. The vehicular demands produced by the travel models are used by the post-processors to compute mean vehicle speeds.

Network
- A highway network is a group of interconnected streets and highways in a study area. A transit network is a group of interconnected transit lines. Travel demand model networks consist of link and node representations of the real world transportation network.

Link
- A segment of street usually with relatively constant demand and capacity characteristics. Travel demand models use links and nodes to represent the highway and transit networks.

Node
- A node is a point representing the intersection of two or more streets (or transit lines).

Speed
- The average rate of travel over a selected course. The average speed of several vehicles over a selected course can be determined by two methods of averaging: averaging speeds, or averaging travel times. This report refers exclusively to the mean speed computed by averaging travel times not by averaging speeds (see May [1] for more complete discussion of difference between space mean speed and time mean speed).

Free-Flow Speed
- The mean vehicular speed on a link at demand levels so low that they do not affect speed. For some facilities, like two-lane roads, this is the speed when only a single vehicle is present. Other facilities, like freeways can accommodate several hundred vehicles per hour at the free-flow speed.

Posted Speed Limit
- The prima-facie speed limit adopted by a jurisdiction for the street.

Capacity
- The maximum sustainable vehicular flow rate for a street usually over a period of one hour. The adjectives "Practical" or "Planning" may be placed in front of the term capacity to indicate a value less than the maximum sustainable flow rate.

Highway Capacity Manual
- A set of equations and procedures for predicting speed and level of service published by the Transportation Research Board.

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VMT	• Number of vehicle-miles traveled. The sum of the number of miles traveled by all the vehicles using the highway network.
VKT	• Number of vehicle-kilometers traveled.
ISTEA	• Intermodal Surface Transportation Efficiency Act.
HPMS	• Highway Performance Management System. HPMS collects volume and design data for selected links in the highway network and extrapolates this data to the entire highway network. This data is used to compute various performance measures (speed, delay, etc.) for the entire network.
NCHRP	• National Cooperative Highway Research Program jointly funded by states and the federal government and operated by the Transportation Research Board.
CAAA	• Clean Air Act Amendments of 1990.
SIP	• State implementation plan for achieving air quality standards.
MOBILE	• An Environmental Protection Agency (EPA) approved set of equations and procedures for estimating vehicular emission rates.
EMFAC	• A California Air Resources Board (CARB) approved set of equations and procedures for estimating vehicular emission rates for vehicles subject to California air pollution control requirements.
FTP	• Federal Test Procedure for determining the pollutant emission rates for vehicles.
Vehicle Operating Mode	• A general characterization of the current operating temperature of the engine of the vehicle. Cold start mode, means the engine was recently started and has not yet warmed up to normal sustained operating temperatures.

Chapter 1. Introduction

Air quality analysis combines travel demand and vehicle emission forecasting. Travel demand models are used to forecast vehicular demand, which air pollutant emission models then use to predict vehicular pollutant emissions. The passage of the Clean Air Act Amendments of 1990 (CAAA) has dramatically increased the importance of this combined travel demand and air pollutant emission forecasting process. Transportation improvement projects cannot proceed without a demonstration of their conformity with state implementation plans (SIP's) to improve air quality.

The weak point in the combined demand/pollution forecasting process has been the interface between the demand forecasting and the pollutant emission prediction steps of the process. Travel demand models are designed to forecast travel demand but have not traditionally been used to forecast speeds. Air pollutant models however require both vehicle demand and speeds, with speed being a crucial factor for estimating emission rates.

Travel demand modelers have attempted to improve their speed prediction capabilities by employing better speed-flow curves or using a post-processor to compute more accurate speeds from the travel demand forecasts.

Air quality forecasters also need to know the proportion of travel made in "cold start" mode, when emission rates are highest. Travel models have not traditionally tracked this kind of information, although recent software improvements now allow the tallying of vehicle-miles traveled (VMT) by vehicle operating mode.

This report summarizes the state of the art in speed prediction for travel demand models and suggests various short term improvements that might be made to the speed estimation routines contained in these models. This report also suggests post-processor routines that can be used to further improve the model speed estimates using data not available to the model. Finally this report suggests improvements that can be made in current techniques for estimating vehicle operating modes (cold start).

1.1 Current Practice - Air Pollutant Estimation

Air pollutant emission models estimate pollutant emissions by multiplying vehicle-miles or vehicle-kilometers traveled (VMT or VKT) by the estimated emission rates. The emission rates are determined based upon ambient conditions, vehicle operating mode, and mean vehicle speed.

Test vehicles are driven through the Federal Test Procedure (FTP) driving cycles to determine the base emission rates by vehicle operating mode (e.g. cold start, etc.). The vehicle operating mode is determined by the amount of time that the vehicle has been in operation. Each FTP driving cycle is characterized by a mean trip speed which embodies a set of assumed accelerations and deceleration events. The base emission rates are adjusted for such factors as speed, temperature, and fuel.

Figure 1 shows how vehicle emission rates vary by mean trip speed when compared to the base emission rate derived according to the FTP driving cycle (CARB [2]). As can be seen, emissions are highly sensitive to the mean speed at very low and very high speeds. An error of 5 mph (8 kph) can affect the emission rate by 50% or more at the low and high ends of the speed range.

Standard practice in emissions estimation is to sum the VMT (VKT) on all the highway links in the study area categorized by the mean speed on each link. The resulting VMT subtotals by speed category are multiplied by the appropriate emission rate for each speed category. DeCorla-Souza [3] however points out that the emission rates were derived based upon average trip speed, not average link speed. DeCorla-Souza found in one example that the traditional link based approach for summing emissions resulted in a 15% higher estimate of hydrocarbon emissions than the trip based approach.

Speed Correction Factors
LDA-CAT - MY 1990

Figure 1. Emission Rate Speed Correction Factors (source: CARB)

In every state, except California, the start emissions are estimated as a part of overall running emissions. The total VMT (VKT) is multiplied by an emission rate that assumes a set of fixed percentages of vehicle travel in cold start mode. Recent research and software improvements however make it possible to estimate the cold start VMT on a link or on a trip purpose basis.

1.2 Current Practice - Speed Estimation

Travel demand models forecast vehicle volumes based upon socio-economic and transportation network data. Speed has traditionally been an input to this process.

Mean vehicle speeds on a road link are estimated using a speed-flow curve that typically relates travel speed to the free-flow speed on the link and the ratio of the predicted traffic volume to the link capacity. The free-flow speed is the speed of travel when demand volumes are too low to affect speed (e.g. one vehicle on the link).

The most commonly used speed-flow curve is the Bureau of Public Roads (BPR) curve. This curve requires as input the free-flow speed, the volume, and the capacity of each highway link.

Metropolitan planning organizations (MPO's) have traditionally not had sufficient resources to measure free-flow speeds or capacities in the field, so these input items are often estimated based upon facility and area type. Link capacities and speeds are often then adjusted in the model (along with demand parameters) as necessary to achieve realistic traffic forecasts.

The traditional BPR curve was derived from the 1965 Highway Capacity Manual, and recent research suggests that it is out of date. The BPR curve under-estimates speeds at volumes approaching capacity and over-estimates speeds when demand exceeds capacity. Additional research suggests that the volume/capacity ratio needs to be supplemented with additional data on signal spacing, signal timing, and signal progression to adequately predict mean vehicle speeds on signalized arterials.
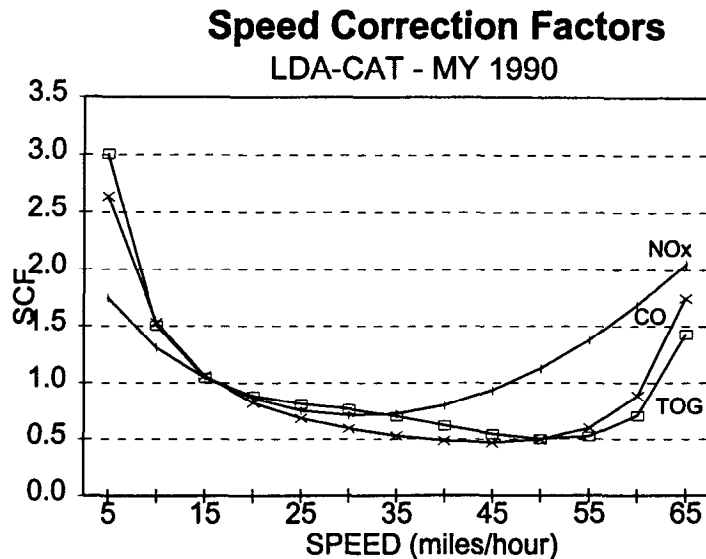
Several MPO's have updated the BPR curve based upon local speed data. They have varied the parameters of the equation to better fit freeway and arterial speed-flow data.

Others have treated speed as more of an input item than an output item. They have adopted speed-flow curves with different forms in order to reduce the computation time needed to assign traffic to the network.

Post processing has been used by a few agencies in special situations where it was desired to perform an analysis of facility operations with hourly demands constrained to hourly capacities and queuing analysis being used to determine delay. Examples are the Central Artery Study for Boston, and the I-710 study in Los Angeles.

1.3 Rationale for Considering Post Processors

This report looks at various techniques for improving the speed estimation procedures within travel demand models. However, there are several constraints on the improvements that can be made within traditional models that make it desirable to also consider the use of post processor algorithms to refine the model estimates after the demand estimation process is complete. These constraints include:

- **Limitations of Existing Software.**

Only a few of the commercially available software packages for creating and operating travel demand models have the capability to incorporate anything but simple link data in the calculation of speeds. Node delays, which become important in the estimation of travel time for signalized facilities, are currently included in only a few software packages. Link interactions and queuing are also excluded from the travel time calculations.

- **Data Requirements**

Most regional planning agencies do not have the resources to assemble the detailed facility data required to accurately estimate speeds. Post-processing allows the agencies to gather the detailed data only for the portion of their network that is the focus of a more detailed study.

- **Model Calibration Requirements**

Many of today's models were calibrated to produce accurate volume forecasts by adjusting free-flow speeds and capacities. The rationale for adjusting these parameters has been that they were rarely measured in the field. Thus they were a likely source of model error.

- **Air Quality Modeling Needs**

Air quality models need demand and speed forecasts on a temporal basis throughout the day. Most travel demand models focus on daily or peak period forecasts. Post processors must be used to expand peak hour forecasts or to allocate daily forecasts to 24 hours of the day.

A major concern with the use of speed post processors is the inconsistency that they necessarily introduce between the speeds used in the model to estimate demand and the speeds produced by the post-processor for use in air quality analysis. A feedback process could overcome this problem but it is computationally difficult to tie a post-processor into the equilibrium assignment process.

1.4 Recommendations

It is recommended that MPO's and other modelers consider implementing one or more of the following options for improving the speed estimation and vehicle operating mode estimation capabilities of travel/air pollutant emission models:

1. Consider replacing the standard BPR curve used in most models with an updated version that shows less impact of volumes on speeds until speeds approach capacity, and then drops very rapidly. This flatter speed-flow curve with a sudden drop at capacity will adversely affect traffic assignment run times by making it more difficult to find equilibrium, but it more accurately reflects true vehicle speeds at capacity and when demand exceeds capacity.

2. Consider enhancing their methods for estimating link free-flow speed and capacity. Recent research found that the accuracy of the standard BPR curve could be improved by 50% by using actual free-flow speeds and capacities rather than estimated values based on area type and facility type. Suggested procedures are provided in this report.

3. Where additional data and resources are available, MPO's should consider using a post processor to estimate link speeds. A post processor allows more explicit treatment of queuing, intersection, and facility operations in the estimating of mean speed.

4. Consider replacing the default operating mode fractions within the MOBILE emissions factor model with updated fractions disaggregated by time of day and trip purpose. These updated mode fractions could be derived from the travel demand model estimates of VMT by operating mode or from survey data.

Transportation modelers should recognize that these proposed enhancements to the speed estimation procedures will result in speed estimates that are much more sensitive to the accuracy of the demand forecasts than for current methods. As demand reaches or exceeds capacity, the estimated mean vehicle speed can fluctuate greatly with minor changes in demand (as it does in real life). Modelers may then need to consider the impacts of this congestion on the spreading of peak hour demand to non-peak hours.

Chapter 2. Improved Speed Models

This chapter reviews current speed estimation practice and describes various methods for improving speed estimation within the traffic assignment portion of a typical travel demand model. The following chapter describes additional methods that can be used outside of the model.

The speed estimation problem consists of three components:

1. Estimation of free-flow speed,

2. Estimation of capacity, and

3. The speed-flow relationship.

The following sections first describe current practice for estimating free-flow speed, capacity, and the speed-flow curve. Then various techniques are presented for improving each of these 3 components of the speed estimation process.

2.1 Current Practice

The vast majority of planning agencies and travel demand model software use the BPR speed-flow curve (or one of its variations) to predict mean vehicle speed. The BPR curve (and its variations) predict speed based upon three pieces of information: the free-flow speed, capacity, and volume. The Standard BPR equation is as follows:

$$s = \frac{s_f}{1 + a(v/c)^b}$$ (eqn. 1)

where:

  s = predicted mean speed

  s_f = free flow speed

  v = volume

  c = practical capacity,

  a = 0.15

  b = 4

Figure 2 Plot of BPR Curve and Some Variations

Practical capacity is defined in this equation as 80% of the capacity. Free-flow speed is defined as 1.15 times the speed at the practical capacity.

The parameter "a" determines the ratio of free-flow speed to the speed at capacity.

The parameter "b" determines how abruptly the curve drops from the free-flow speed. A high value of "b" causes speed to be insensitive to v/c until the v/c gets close to 1.0, then the speed drops abruptly.

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Planning agencies typically use "look-up" tables for link capacity and free-flow speed. Table 1 and Table 2 illustrate typical "look-up" tables for capacity and free-flow speed. Many agencies develop their own local look-up tables.

Table 1. Practical Capacity Look-Up Table for BPR Curve						
One-Way Level of Service "C" Vehicles Per Lane Per Hour (VPH)						
	Freeway	Express-way	2-Way Arterial (Parking)	One-Way Arterial (Parking)	Centroid Connector	2-Way Arterial (No Park)
CBD	1750	800	600	700	10,000	600
Fringe	1750	1000	550	550	10,000	800
Outer CBD	1750	1000	550	650	10,000	800
Rural/ Residential	1750	1100	550	900	10,000	800

Source: Comsis [4]

Table 2. Free-Flow Speed Look-Up Table For BPR Curve						
Free-Flow Speeds (MPH)						
	Freeway	Expresswa y	2-Way Arterial (Parking)	One-Way Arterial (Parking)	Centroid Connector	2-Way Arterial (No Park)
CBD	48	37	22	22	10	22
Fringe	48	44	25	29	15	25
Outer CBD	58	37	22	24	15	22
Rural/ Residential	67	47	28	32	15	28

Source: Comsis [5]

The current BPR curve has several weaknesses.

The accuracy of the BPR curve hinges upon the accuracy of the free-flow speed and capacity estimates used as input to the method. Recent research by Dowling [6] found that the use of actual link capacities and free flow speeds rather than the default estimates contained in the above look-up tables can reduce the BPR's speed estimation error by 50%.

The BPR curve was fit to freeway speed-flow data used to develop the 1965 Highway Capacity Manual. At that time it was believed that the flow-density relationship for a freeway was a polynomial curve. Recent data, upon which the current Highway Capacity Manual (HCM) [7] is based, shows a flat speed-flow relationship for freeways (speed is insensitive to flow) until flow approaches capacity.

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Speed-Flow Characteristics for 4-Lane Freeways

(Figure 3-2 Highway Capacity Manual)
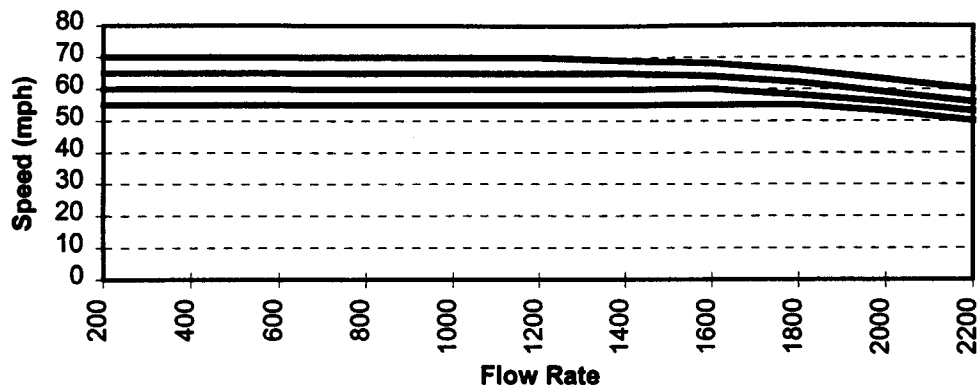
Figure 3. Speed -Flow Curves for 4-Lane Freeway (source: Highway Capacity Manual)

The standard BPR curve also under predicts the delays associated with congestion.

Finally, the BPR curve does not include significant variables that affect travel time on signalized arterials The volume/capacity (v/c) ratio in the real world has little influence on travel time (until volume exceeds capacity). Figure 4 shows a plot of observed mean vehicle speeds against the BPR curve for Ventura Boulevard in Los Angeles California. The variance in speeds at a given v/c ratio is greater than the variation in the mean speed over the range of v/c ratios[1].

Ventura Boulevard, Los Angeles, CA
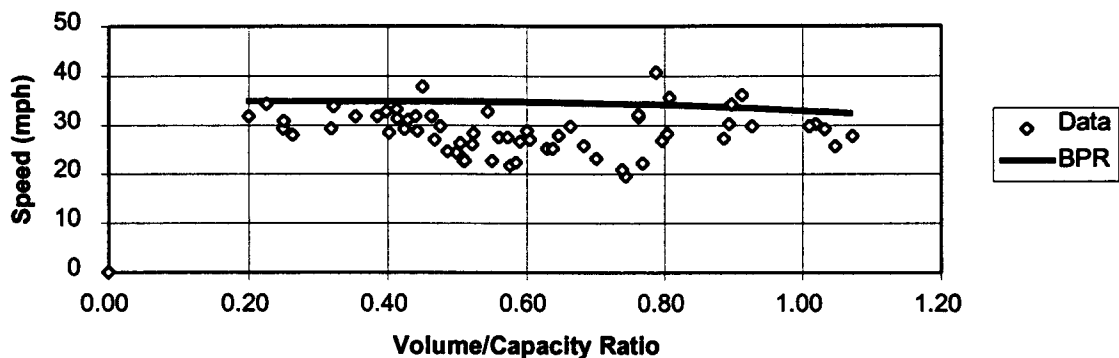
Figure 4. Standard BPR Curve Versus Arterial Speed Data (source: Dowling - NCHRP 3-55(2))

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[1] Volumes and speeds are for 15 minute intervals measured 7AM to 9 AM at loops on approach to each intersection on December 7, 1993. Capacities for each approach estimated using 1600 vehicles per hour per lane ideal saturation flow. The v/c ratio may exceed 1.00 because estimated rather than actual saturation flow rates are used to estimate capacity.

Many MPO's have been concerned about inaccuracies in the speeds estimated by the standard BPR curve. They have updated the basic BPR curve based upon more recent data in the 1985 HCM or locally collected speed-flow data. The updated BPR curves have "a" parameters that vary from 0.1 to 1.0, and "b" (power) parameters that vary from 4 to 11. These agencies have also constructed their own local default look-up tables of practical capacity and free-flow speeds.

One area, Dallas-Forth Worth, uses an exponential equation instead of the standard polynomial form.

Others have been concerned about the very low speeds predicted at extremely high v/c ratios[2]. These agencies use an updated version of the BPR curve for v/c ratios less than a certain limit (usually between 1.33 and 2.00). They use a completely different equation for the higher v/c ratios. These "split" equations are designed to expedite the rate of closure for the traffic assignment algorithm. Extremely low speeds at high volume/capacity ratios tend to cause the traffic assignment results to fluctuate wildly between iterations.

Singh [8] is an excellent reference on many of the adaptations that have been made by planning agencies to improve the performance of the BPR curve for local conditions.

2.2 Improved Free-Flow Speed Estimation Techniques

This section presents potential techniques for better estimating link free-flow speeds. The first few techniques start with an "ideal" free-flow speed which is then adjusted downwards based upon geometric conditions. The last technique, NCHRP 3-55(2), estimates free-flow speed based upon the posted speed limit and signal control data.

2.2.1 Highway Capacity Manual Methods

The Highway Capacity Manual (HCM) [9] provides simple procedures for estimating facility free-flow speeds for freeways, multi-lane rural highways, and signalized urban arterials. These procedures however often require geometric data for the facility that is difficult to obtain for most planning agencies. The authors are not aware of any actual applications of the HCM free-flow speed equations in travel demand modeling practice.

Freeways

The 1997 update of Chapter 3 of the HCM provides an equation for estimating the freeway free-flow speed based on the number of lanes, the lane width, lateral clearance, and the number of interchanges per mile (see Schoen [10]). The formula assumes that the ideal free-flow speed is 70 mph (112 kph) before reduction for the factors listed above.

$$FFS = 70 - F_n - F_{lw} - F_{lc} - F_{id} \qquad \text{(eqn. 2)}$$

where:
FFS = free flow speed for basic freeway segment (mph)
F_n = Adjustment factor for effect of number of lanes.
F_{lw} = Adjustment factor for effect of lane width.
F_{lc} = Adjustment factor for effect of lateral clearance.
F_{id} = Adjustment factor for effect of interchange density.

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[2] Technically, once the v/c ratio exceeds 1.00, it is no longer a measurable hourly volume, but a demand. Some researchers use d/c ratio to distinguish between measurable volume and a demand.

Multi-Lane Highways

Chapter 7 of the HCM provides a procedure for computing the "actual" free-flow speed given the median type, the lane width, lateral clearance, and number of access points per mile (see Equation 7-1, Table 7-2, Table 7-3, Table 7-4, Table 7-5 of the HCM). Unfortunately, the user must also provide the "ideal" free-flow speed before these adjustments can be applied to arrive at the "actual" free-flow speed. Page 7-10 of the HCM cites recent research (no references provided) that found that ideal free flow speed is 5 to 7 mph higher than the posted speed limit.

$$FFS = FFS_I - F_M - F_{LW} - F_{LC} - F_A \qquad \text{(eqn. 3)}$$

where:
FFS = Computed Free-Flow Speed (mph),
FFS_I = Ideal free-flow speed (mph),
F_M = Adjustment factor for median type,
F_{LW} = Lane width adjustment,
F_{LC} = Lateral clearance adjustment,
F_A = Access points density adjustment.

Urban Arterials

Chapter 11 of the Highway Capacity Manual provides a look-up table for converting mid-block free-flow speed on a signalized arterial to segment running times per mile (exclusive of delays at signals). The average running speed between signals is a function of the spacing of the traffic signals. The mid-block free-flow speed is related to the arterial class. There are three classes of arterials in the HCM.

The following equation for computing the running speed between signals has been fitted by Dowling to the HCM table:

$$S = s_f - A * \exp(B * dist) \qquad \text{(eqn. 4)}$$

$$\text{where: } A = 18 + \frac{s_f - 25}{2.22} \qquad \text{(eqn. 5)}$$

$$\text{where: } B = \frac{s_f - 25}{5} - 9 \qquad \text{(eqn. 6)}$$

and where:

S   = Mean running speed between signals (mph)
s_f = mid-block free-flow speed (mph)
dist.   = Average Distance between signals (miles)

2.2.2 NCHRP 3-55(2) Method

Dowling [11] suggests a set of linear equations for estimating free-flow speed based upon data gathered on mean speed, and the posted speed limit. Two regression equations are recommended, one for high speed facilities (speed greater than 50 mph), the other for lower speed facilities. These equations were derived from field measurements of free-flow speed, but have not yet been tested in actual travel demand modeling practice.

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The following linear equation was fitted to rural freeway data obtained as part of the NCHRP research project. A total of 10 data points were obtained for 6 facilities in Oregon, California, and New Hampshire. The data set is relatively limited in range (most facilities had posted speed limits of 55 mph (88 kph), with a few with posted limits of 65 mph (104 kph). The equation appears valid for facilities with posted speed limits in excess of 50 mph (80 kph).

Customary Units:

$$\text{Mean Speed (mph)} = 0.88 * (\text{the Posted Speed Limit in mph}) + 14 \qquad \text{(eqn. 7)}$$

SI Units:

$$\text{Mean Speed (kph)} = 0.88 * (\text{the Posted Speed Limit in kph}) + 22 \qquad \text{(eqn. 8)}$$

Florida DOT personnel have suggested that simply adding 5 to 7 mph (8 to 11 kph) to the posted freeway speed limit would probably be as accurate as using the above linear regression curves.

The following linear equation was fitted by Dowling to a data set gathered by Tignor & Warren [12]. This equation appears to be valid for expressways and the mid-block points on signalized arterials where the posted speed limit is 50 mph or less.

Customary Units:

$$\text{Mean Speed (mph)} = 0.79 * (\text{the Posted Speed Limit in mph}) + 12 \qquad \text{(eqn. 9)}$$

SI Units:

$$\text{Mean Speed (kph)} = 0.79 * (\text{the Posted Speed Limit in kph}) + 19 \qquad \text{(eqn. 10)}$$
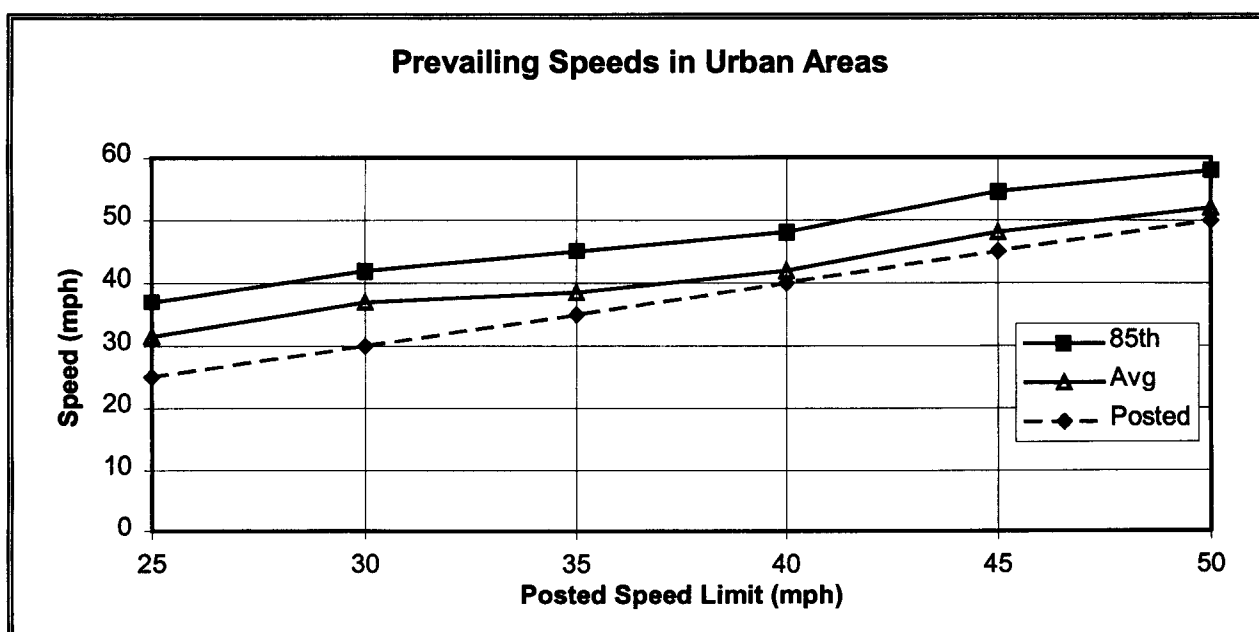
Figure 5. Speed Limit, 85 Percentile, and Mean Speeds in Urban Areas. (Tignor & Warren)

Dowling suggests the following equations for estimating the free-flow speed for signalized facilities that takes into account both the posted speed limit and the signal delays along the street (which occur even at

low volumes). The mean free flow speed (including signal delay) is computed using the following equation which adds together the free-flow travel time between signals and the delay time at signals (under free-flow conditions).

$$S_f = \frac{L}{L/S_{mb} + N*(D/3600)}$$

(eqn. 11)

Where:

S_f = Free flow speed for urban interrupted facility (mph or kph)

L = Length of facility (miles or km)

S_{mb} = mid-block free flow speed (mph or kph)

= 0.79 (Posted Speed Limit in mph) + 12 (mph)

= 0.79 (Posted Speed Limit in kph) + 19 (kph)

N = number of signalized intersections on length "L" of facility

D = average delay per signal per equation 4 below (sec).

The average delay per signal is computed using the following equation:

D = DF * 0.5 * C(1-g/C)^2

(eqn. 12)

where:

D = The total signal delay per vehicle (sec)

g = The effective green time (sec)

C = The cycle length (sec)

If signal timing data is not available, the planner can use local customary values or the following default values:
C = 120 seconds
g/C = 0.45

DF = (1-P) / (1-g/C) where: P = The proportion of vehicles arriving on green

If "P" is unknown, the following defaults can be used for "DF":

DF = 0.9 for uncoordinated traffic actuated signals

= 1.0 for uncoordinated fixed time signals

= 1.2 for coordinated signals with unfavorable progression

= 0.90 for coordinated signals with favorable progression

= 0.60 for coordinated signals with highly favorable progression

2.2.3 Evaluation of Improved Free-Flow Speed Estimation Techniques

The Highway Capacity Manual techniques allow the planner to estimate the speed reducing effects of geometric design factors and access point density. Planners however rarely have access to the necessary geometric design details such as lane width and lateral clearance. The HCM techniques are also currently limited to a specific set of facility types.

The NCHRP 3-55(2) method can be applied to any facility where the posted speed limit is known. However, the method is not reliable if local agencies have used "atypical" criteria for setting the speed limits. The 85 percentile speed should then be used instead of the posted speed limit. The HCM techniques may be used where the additional data is available to the planner.

Readers should note that both techniques are expected to yield more accurate estimates of free-flow speeds than current practice using look-up tables. However, neither of these techniques have been employed in actual travel demand modeling practice, and thus their impact on model operation and results is unknown.

2.3 Improved Capacity Estimation Techniques

This section presents candidate techniques for improving the estimation of link capacities. The Highway Capacity Manual (HCM) is the most generally accepted method for estimating capacity, however; its procedures require a great deal of data not readily available to planning agencies. Florida and NCHRP 3-55(2) suggest two approaches for applying the HCM methods when the necessary data is not readily available.

2.3.1 Highway Capacity Manual

The Highway Capacity Manual (HCM) provides an extensive set of techniques for estimating the capacity of freeways, multi-lane highways, and two lane rural roads. The HCM however does not provide a technique for estimating the link capacity for signalized facilities (facilities where signals are spaced 2 miles (3.2 km) or less apart). In this case, the HCM provides techniques for estimating node capacities.

The HCM techniques tend to focus on converting real world traffic volumes into ideal flow rates. The ideal flow rates are then compared to the ideal capacity of a facility to obtain the volume/capacity ratio. This unfortunately results in the creation of artificial flows for the purpose of capacity analysis.

The ideal capacities for freeways are 2300 vph/lane for 6-lane freeways, and 2200 vph/lane for 4-lane freeways. Schoen recommends that these ideal capacities be replaced with a range of ideal capacities that decrease from 2400 to 2250 vph/lane as the free-flow speed decreases.

The HCM recommends ideal capacities for multi-lane highways that range from 1900 to 2200 vph/lane, depending on the free-flow speed. The highest capacity is associated with the highest free-flow speed (60 mph, 97 kph).

The HCM recommends an ideal capacity of 2800 vehicles per hour, total of both directions, for two-lane rural roads.

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2.3.2 Florida DOT Method

The Florida Department of Transportation (FDOT) developed a Level of Service Manual [13] which contains planning techniques and tables for estimating maximum service flow rates using data typically available to planners. The techniques are based upon the Highway Capacity Manual, substituting defaults for some of the more difficult to obtain input data for the HCM methods.

The Florida LOS Manual consists of generalized level of service tables that planners can look-up to find the maximum service volume, and software that planners can use to create customized service volumes for specific facility characteristics and areas. The software consists of spreadsheets that can be used to create tables of average service volumes for the entire facility and a software implementation of the Chapter 11 of the Highway Capacity Manual (ARTPLAN).

The generalized level of service tables in the Florida LOS Manual provide maximum service volumes by facility type and general characteristics for four area types: Urbanized Areas, Transition Areas, Developed Places (less than 5,000 population), and Undeveloped Rural Areas.

The tables were generated using the 1994 HCM methodology and sets of agreed upon assumptions for each facility type and area type. The assumptions are averages for the entire facility being analyzed, and do not take into account certain unusual facility characteristics, or special problem spots within a facility.

The following tables show the FDOT Generalized Level of Service Tables and the assumptions used by FDOT to generate these tables. The tables were generated by creating different sets of default input values for each facility and area type, and substituting these defaults into the Highway Capacity Manual methods. Readers should note that FDOT recommends that planners compute the service volumes using the FDOT software and the specific characteristics of the facility rather than relying on the Generalized Level of Service Tables. The tables are to be used only for preliminary estimates.

Table 3. Florida Generalized Peak Hour Directional Volumes for Urbanized Areas							
			Level of Service				
Facility	lanes	Divided?	A	B	C	D	E
Freeways	4	n/a	1100	1760	2640	3350	4040
(Group 1)[3]	6	n/a	1660	2640	3970	5030	6340
	8	n/a	2210	3530	5290	6700	8460
	10	n/a	2760	4410	6620	8380	10570
Freeways	4	n/a	1060	1700	2550	3230	3900
(Group 2)[4]	6	n/a	1600	2560	3840	4860	6130
	8	n/a	2130	3410	5110	6480	8170
	10	n/a	2670	4260	6390	8100	10210
State	2	No	460	720	980	1280	1710
Multi-lane	4	Yes	1110	1850	2590	3110	3700
Highways	6	Yes	1670	2780	3890	4660	5550
Class Ia[5]	2	No	*	660	810	880	900
Interrupted	4	Yes	*	1470	1760	1890	1890
Flow	6	Yes	*	2280	2660	2840	2840
	8	Yes	*	2840	3280	3480	3480
Class Ib[6]	2	No	*	*	460	760	840
Interrupted	4	Yes	*	*	1020	1640	1800
Flow	6	Yes	*	*	1550	2510	2710
	8	Yes	*	*	1890	3060	3320
Class II[7]	2	No	*	*	*	620	800
Interrupted	4	Yes	*	*	*	1390	1740
Flow	6	Yes	*	*	*	2130	2640
	8	Yes	*	*	*	2600	3230
Class III[8]	2	No	*	*	*	690	780
Interrupted	4	Yes	*	*	*	1540	1700
Flow	6	Yes	*	*	*	2340	2570
	8	Yes	*	*	*	2860	3140

n/a = not applicable.   * = Level of service cannot be achieved.

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[3] Group 1 freeways are located within an urbanized area with over 500,000 population and the freeways lead to or are within 5 miles of the primary Central Business District.

[4] Group 2 freeways are freeways not falling within Group 1.

[5] Class Ia arterials have less than 2.50 signals per mile.

[6] Class Ib arterials have 2.50 to 4.50 signals per mile.

[7] Class II arterials have more than 4.50 signals per mile and are NOT located within a primary central business district of an urbanized area with over 500,000 population.

[8] Class III arterials have more than 4.50 signals per mile AND are located within the primary central business district of an urbanized area with over 500,000 population.

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Table 4. Adjustments for Divided/Undivided Streets and Left Turn Bays

Adjust the maximum service volumes by the following percentages			
Lanes	Median	Left Turn Bays	Adjustment Factors
2	Divided	Yes	+5%
2	Undivided	No	-20%
Multi	Undivided	Yes	-5%
Multi	Undivided	No	-25%

Table 5. Adjustments For One-Way Streets

Adjust the maximum service volumes by the following percentages		
One-Way Lanes	Corresponding Two-Way Lanes	Adjustment Factor
2	4	+20%
3	6	+20%
4	8	+20%
5	8	+50%

Table 6. Default Input Values for Urbanized Areas

Input Data	Freeways		State Two-Way Arterials				
	Group 1	Group 2	Uninter-rupted	Class 1a	Class 1b	Class 2	Class 3
Traffic Characteristics							
Peak Hour Factor	0.950	0.950	0.925	0.925	0.925	0.925	0.925
Adjt. Sat. Flow Rate							
2-lane facility	2125	2050	1850	1850	1850	1850	1800
4-6 lanes	2225	2150	2000	1850	1850	1850	1800
8 lanes	2225	2150	NA	1700	1700	1700	1650
10 lanes	2225	2150	NA	NA	NA	NA	NA
Turn from excl. lane	NA	NA	NA	0.12	0.12	0.12	0.12
Road Characteristics							
Through Lanes	4-12	4-12	2-6	2-8	2-8	2-8	2-8
Arterial Class.	NA	NA	NA	I	I	II	III
Free-flow speed	60	60	50	45	40	35	30
Medians	Yes	Yes	Yes	Yes	Yes	Yes	Yes
left turn bays	NA	NA	Yes	Yes	Yes	Yes	Yes
Signal Characteristics							
Signals per mile	NA	NA	NA	1.5	3.0	5.0	7.5
Arrival type	NA	NA	NA	3	4	4	4
Signal Type	NA	NA	NA	Act	Semi	Semi	Semi
Cycle length	NA	NA	NA	120	120	120	120
Weighted effect. g/c	NA	NA	NA	0.45	0.45	0.45	0.45

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Table 7. Florida Generalized Peak Hour Directional Volumes for Areas Transitioning into Urbanized Areas, or Areas Over 5,000 Population Not in Urbanized Area

Facility	lanes	Divided?	Level of Service A	B	C	D	E
Freeways	4	n/a	1110	1770	2640	3330	3750
	6	n/a	1670	2670	3980	5020	5910
	8	n/a	2230	3560	5310	6690	7890
	10	n/a	2790	4460	6630	8370	9860
State	2	No	440	690	930	1230	1640
Multi-lane	4	Yes	1090	1820	2520	3010	3500
Highways[9]	6	Yes	1630	2730	3780	4520	5260
Major	2	No	*	*	520	680	750
City/County	4	Yes	*	*	1170	1490	1600
Roadways	6	Yes	*	*	1810	2280	2410
Class Ia[10]	2	No	*	610	750	820	850
Interrupted	4	Yes	*	1360	1640	1750	1790
Flow	6	Yes	*	2110	2480	2650	2680
Class Ib[11]	2	No	*	*	430	700	780
Interrupted	4	Yes	*	*	940	1530	1670
Flow	6	Yes	*	*	1440	2330	2520
Class II[12]	2	No	*	*	*	570	740
Interrupted	4	Yes	*	*	*	1280	1620
Flow	6	Yes	*	*	*	1980	2460
Other	2	No	*	*	250	490	560
Signalized[13]	4	Yes	*	*	540	1080	1210

n/a = not applicable.   * = Level of service cannot be achieved.

Table 8. Adjustments for Divided/Undivided Streets and Left Turn Bays (Transitioning Areas)

Adjust the maximum service volumes by the following percentages

Lanes	Median	Left Turn Bays	Adjustment Factors
2	Divided	Yes	+5%
2	Undivided	No	-20%
Multi	Undivided	Yes	-5%
Multi	Undivided	No	-25%

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[9] Signals, if any, spaced more than two miles apart (0.5 signals per mile).

[10] Class Ia arterials have less than 2.50 signals per mile.

[11] Class Ib arterials have 2.50 to 4.50 signals per mile.

[12] Class II arterials have more than 4.50 signals per mile and are NOT located within a primary central business district of an urbanized area with over 500,000 population.

[13] Class I and Class II arterials are generally state highways. This category allows for minor roads that are not state highways.

Table 9. Adjustments For One-Way Streets (Transitioning Areas)

Adjust the maximum service volumes by the following percentages

One-Way Lanes	Corresponding Two-Way Lanes	Adjustment Factor
2	4	+20%
3	6	+20%
4	6	+50%

Table 10. Default Input Values for Transitioning Areas

Input Data	Freeways	State Two-Way Arterials				Non-State	
		Uninterrupted	Class 1a	Class 1b	Class 2	Major	Other
Traffic Characteristics							
Peak Hour Factor	0.950	0.910	0.910	0.910	0.910	0.925	0.910
Adjusted Sat. Flow Rate							
2-lane facility	1975	1800	1750	1750	1750	1750	1700
4 lanes	2075	1925	1750	1750	1750	1750	1700
6 lanes	2075	1925	1750	1750	1750	1750	NA
Turn from excl. lane	NA	NA	0.12	0.12	0.12	0.12	0.16
Road Characteristics							
Through Lanes	4-82	2-6	2-6	2-6	2-6	2-6	2-6
Arterial Class.	NA	NA	I	I	II	I	NA
Free-flow speed	65	55	45	40	35	45	NA
Medians	Yes	Yes	Yes	Yes	Yes	Yes	Yes
left turn bays	NA	Yes	Yes	Yes	Yes	Yes	Yes
Signal Characteristics							
Signals per mile	NA	NA	1.5	3.0	5.0	2.5	NA
Arrival type	NA	NA	3	4	4	4	3
Signal Type	NA	NA	Act	Semi	Semi	Semi	Semi
Cycle length	NA	NA	120	120	120	120	120
Weighted effect. g/c	NA	NA	0.45	0.45	0.45	0.42	0.32

Table 11. Florida Generalized Peak Hour Directional Volumes for Cities or Developed Areas With Less than 5,000 Population (not in Urbanized Area)							
			Level of Service				
Facility	**lanes**	**Divided?**	**A**	**B**	**C**	**D**	**E**
Freeways	4	n/a	1150	1840	2700	3310	3610
	6	n/a	1730	2780	4080	4990	5700
	8	n/a	2310	3700	5430	6660	7600
State	4	undiv, no bays	770	1290	1790	2140	2480
Multi-lane	4	undiv, yes bays	980	1640	2270	2710	3150
Highways[14]	4	divided, yes bays	1030	1720	2380	2850	3310
55 mph Speed Limit	6	divided, yes bays	1540	2580	3580	4270	4970
State	4	undiv, no bays	700	1170	1640	1960	2480
Multi-lane	4	undiv, yes bays	880	1480	2080	2490	3150
Highways	4	divided, yes bays	930	1560	2190	2620	3310
45 mph Speed Limit	6	divided, yes bays	1390	2330	3280	3920	4970
State 2-Lane	2	undiv, no bays	230	400	570	800	1150
Highways[15]	2	undiv, yes bays	290	500	720	1000	1430
55 mph Speed Limit	2	divided, yes bays	300	530	750	1050	1500
State 2-Lane	2	undiv, no bays	*	330	490	720	1070
Highways	2	undiv, yes bays	*	420	620	900	1340
45 mph Speed Limit	2	divided, yes bays	*	440	650	940	1410
Class Ia[16]	2	undiv, no bays	*	530	590	640	650
Interrupted	2	undiv, yes bays	*	670	750	810	830
Flow	2	divided, yes bays	*	700	790	860	870
	4	undiv, no bays	*	1090	1210	1300	1300
	4	undiv, yes bays	*	1380	1530	1650	1650
	4	divided, yes bays	*	1450	1610	1730	1740
	6	divided, yes bays	*	2230	2440	2610	2610
Class Ia2[17]	2	undiv, no bays	*	*	500	570	610
Interrupted	2	undiv, yes bays	*	*	640	720	780
Flow	2	divided, yes bays	*	*	670	760	820
	4	undiv, no bays	*	*	1140	1260	1330
	4	undiv, yes bays	*	*	1350	1490	1580
	4	divided, yes bays	*	*	1430	1570	1670
	6	divided, yes bays	*	*	2180	2380	2520
Other	2	No Bays	*	*	270	350	400
Signalized[18]	2	Yes Bays	*	*	350	440	500

n/a = not applicable.   * = Level of service cannot be achieved.   Bays = left turn bays.

For passing lane adjustments see table following rural areas.

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[14] Signals, if any, spaced more than two miles apart (0.5 signals per mile).

[15] Less than 0.5 signals per mile (more than 2 miles between signals, if any).

[16] Class Ia1 arterials have 1.50 or less signals per mile.

[17] Class Ia2 arterials have more than 1.50 signals per mile.

[18] Class I and Class II arterials are generally state highways. This category allows for minor roads that are not state highways.

Input Data	Freeways	Multi-Ln	2-Lane 55 mph	2-Lane 45 mph	Interrupt. Class 1a1	Interrrupt Class 1a2	Other Signal
Table 12. Default Input Values for Cities or Developed Areas With Less than 5,000 Population (not in Urbanized Area)							
Traffic Characteristics							
Peak Hour Factor	0.950	0.895	0.895	0.895	0.895	0.895	0.895
Adjust. Saturation. Flow							
2-lane facility	NA	NA	1600	1500	1700	1700	1700
4 lanes	1900	1850	NA	NA	1700	1700	NA
6 lanes	2000	1850	NA	NA	1700	1700	NA
8 lanes	2000	NA	NA	NA	NA	NA	NA
Turn from excl. lane	NA	NA	NA	NA	0.12	0.12	0.16
Road Characteristics							
Through Lanes	4-8	4-6	2	2	2-6	2-6	2
Arterial Class.	NA	NA	NA	NA	I	I	NA
Free-flow speed	70	45/55	55	45	40	35	NA
Medians	Yes	Varies	Varies	Varies	Varies	Varies	No
left turn bays	NA	Varies	Varies	Varies	Varies	Varies	Varies
Signal Characteristics							
Signals per mile	NA	NA	NA	NA	1.0	2.0	NA
Arrival type	NA	NA	NA	NA	3	3	3
Signal Type	NA	NA	NA	NA	Act.	Act.	Act.
Cycle length	NA	NA	NA	NA	120	120	120
Weighted effect. g/c	NA	NA	NA	NA	0.45	0.45	0.32

Table 13. Florida Generalized Peak Hour Directional Volumes for Rural Undeveloped Areas

Facility	lanes	Divided?	Level of Service				
			A	B	C	D	E
Freeways	4	n/a	1150	1840	2700	3310	3610
	6	n/a	1730	2780	4080	4990	5700
	8	n/a	2310	3700	5430	6660	7600
State	4	No, No Bays	810	1340	1830	2170	2440
Multi-lane	4	No, Yes Bays	1020	1700	2320	2750	3090
Highways[19]	4	Yes	1070	1790	2440	2900	3260
	6	Yes	1610	2690	3660	4350	4880
Two-Lane	2	No Bays	140	280	460	740	1190
Highways[20]	2	Bays	150	300	490	770	1250
55mph Posted Speed							
Two-Lane	2	No Bays	*	140	370	600	1140
Highways[21]	2	Bays	*	140	380	640	1200
45 mph Posted Speed							

n/a = not applicable.   * = Level of service cannot be achieved.

Table 14. Adjustments for Passing Lanes

Adjust the maximum service volumes by the following percentages	
Percent Miles with Exclusive Passing Lanes	Adjustment Factors
60%+	+30%
20% - 59%	+20%
5% - 19%	+10%
1% - 4%	+5%

---

[19] Less than 0.5 signals per mile, if any.

[20] Less than 0.5 signals per mile, if any.

[21] Less than 0.5 signals per mile, if any.

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Table 15. Default Input Values for Rural Undeveloped Areas				
			55 mph	45 mph
	Freeways	Multi-Ln	2-Lanes	2-lanes
Traffic Characteristics				
Peak Hour Factor	0.950	0.880	0.880	0.880
Adjt. Sat. Flow Rate				
2-lane facility	NA	NA	2600	2600
4 lanes	1900	1850	NA	NA
6 lanes	2000	1850	NA	NA
8 lanes	2000	NA	NA	NA
Turn from excl. lane	NA	NA	NA	NA
Road Characteristics				
Through Lanes	4-8	4-6	2	2
Arterial Class.	NA	NA	NA	NA
Free-flow speed	70	60	50	45
Medians	Yes	Varies	No	No
left turn bays	NA	Varies	Varies	Varies
Percent No Pass	NA	NA	20%	40%
% Exclusive Pass Lanes	NA	NA	0%	0%
Signal Characteristics				
Signals per mile	NA	NA	NA	NA
Arrival type	NA	NA	NA	NA
Signal Type	NA	NA	NA	NA
Cycle length	NA	NA	NA	NA
Weighted effect. g/c	NA	NA	NA	NA

2.3.3 NCHRP 3-55(2) Method

The NCHRP 3-55(2) method takes the Highway Capacity Manual equations for converting actual volumes to ideal flow rates and translates these equations into equivalent adjustments to the HCM ideal capacities. Most HCM adjustments are preserved with recommended default values (most taken from the FDOT manual) provided for cases where local data is not available to the planner.

Freeways

The following equation is used to compute the capacity of a segment of freeway:

$$\text{Capacity (vph)} = \text{Ideal Cap} * N * F_{hv} * PHF \qquad \text{(eqn. 13)}$$

where:

> Ideal Cap = 2400 passenger cars per hour per lane (pcphl) for freeways with 70 mph (110 kph) or greater free-flow speed.
>
> = 2300 (pcphl) for all other freeways (free flow speed < 70 mph (110 kph)).
>
> N = Number of through lanes.
>
> F_{hv} = Heavy vehicle adjustment factor.
>
> PHF = Peak hour factor.

Unsignalized Multi-Lane Roads

The following equation is used to compute the capacity of a multi-lane road with signals (if any) spaced more than 2 miles apart:

$$\text{Capacity (vph)} = \text{Ideal Cap} * N * F_{hv} * PHF \qquad \text{(eqn. 14)}$$

where:

> Ideal Cap = 2200 (pcphl) for multi-lane rural roads with 60 mph free-flow speed.
>
> = 2100 (pcphl) for multi-lane rural roads with 55 mph free-flow speed.
>
> = 2000 (pcphl) for multi-lane rural roads with 50 mph free-flow speed.
>
> N = Number of through lanes. Ignore exclusive turn lanes.
>
> F_{hv} = Heavy vehicle adjustment factor.
>
> PHF = Peak hour factor.

Two-Lane Unsignalized Roads

The following equation is used to compute the capacity (in one direction) for a two-lane (total of both directions) road with signals (if any) more than 2 miles apart:

$$\text{Capacity (vph)} = \text{Ideal Cap} * N * F_{w} * F_{hv} * PHF * F_{dir} * F_{nopass} \qquad \text{(eqn. 15)}$$

where:

> Ideal Cap = 1400 (pcphl) for all two-lane rural roads.

F_w = Lane width and lateral clearance factor.

F_{hv} = Heavy vehicle adjustment factor.

PHF   = Peak hour factor.

F_{dir} = Directional Adjustment Factor.

F_{nopass} = No-Passing Zone Factor.

Signalized Arterials

The following equation is used to compute the one direction capacity of any signalized road with signals spaced 2 miles or less apart:

Capacity (vph) = Ideal Sat * N * F_{hv} * PHF * F_{park} * F_{Bay} * F_{CBD} * g/C * Fc   (eqn. 16)

where:

Ideal Sat   = Ideal saturation flow rate (vehicles per lane per hour of green).

N   = Number of lanes

F_{hv} = Heavy vehicle adjustment factor

PHF   = Peak hour factor

F_{park} = On-street parking adjustment factor.

F_{Bay} = Left turn bay adjustment factor

F_{CBD} =   = Central Business District (CBD) Adjustment Factor

g/C   = Ratio of effective green time per cycle.

Fc   = Optional user specified calibration factor necessary to match estimated capacity with field measurements or other independent estimates of capacity (no units). Can be used to account for the capacity reducing effects of left and right turns made from through lanes.

All of these equations require facility specific geometric data not commonly available to planning agencies, so as in the Florida Method, it is recommended that planners develop look-up tables of default values for this data according to the area type and facility type. The following two tables show a procedure for selecting default values and computing a look-up table of capacities by facility type, area type, and terrain type. Other classification schemes may be appropriate, depending the nature of local roadway conditions.

Table 16 shows a set of selected default parameters for the calculation of capacity for freeways, divided arterials, undivided arterials, and collectors. Each facility type is further subclassified according to the area type (urban or rural), terrain type (level, rolling, mountainous), and number of lanes (total of 2 lanes both directions, or more). A separate set of default parameters is then selected for each subclassification of each facility type.

For example: a rural freeway in level or rolling terrain is assumed to have a free-flow speed in excess of 70 mph (112 kph), 5% heavy vehicles, and a peak hour factor of 0.85. An urban freeway is assumed to have a free-flow speed below 70 mph (112 kph), 2% heavy vehicles, and a peak hour factor of 0.90 to reflect the lower design speeds, heavier passenger car volumes, and flatter peak volumes in urban areas.

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Divided arterials in rural areas are assumed to have free-flow speeds that decrease as the difficulty of the terrain increases. The assumed free-flow speed for level terrain is 60 mph (96 kph), for rolling terrain it is 55 mph (88 kph), and for mountainous terrain it is 50 mph (80 kph).

Any road in a rural area is assumed in this table to have signals (if any) spaced farther than 2 miles apart. Urban area roads are assumed in this table to have signals at least 2 miles apart. The local planning agency should modify these assumptions if they are not appropriate for its particular jurisdiction.

This table shows assumptions only for 2 lane rural undivided arterials, but the planning agency can add additional rows of data for multi-lane rural undivided arterials.

Table 16
Example Table For Entering Default Values
for Computing Capacity by Functional Class and Area/Terrain Type

Functional Class	Area Type	Terrain Type	Lanes	Free Speed	Lane Width	PHF	% Heavy Vehicles	Direction Split	% No Pass	Parking	Left Turn Bay	g/C
Freeway	Rural	Level	all	> 70 mph		0.85	5%					
		Rolling	all	> 70 mph		0.85	5%					
		Mountain	all	< 70 mph		0.85	5%					
	Urban	all	all	< 70 mph		0.90	2%					
Divided Arterial	Rural	Level	>2	60 mph		0.85	5%					
		Rolling	>2	55 mph		0.85	5%					
		Mountain	>2	50 mph		0.85	5%					
	Suburb	all	all			0.90	2%			no	yes	0.45
	Urban	all	all			0.90	2%			yes	yes	0.45
	CBD	all	all			0.90	2%			yes	yes	0.45
Undivided Arterial	Rural	Level	2		standard	0.85	5%	55%	0%			
		Rolling	2		standard	0.85	5%	55%	60%			
		Mountain	2		narrow	0.85	5%	55%	80%			
	Suburb	all	all			0.90	2%			no	no	0.45
	Urban	all	all			0.90	2%			yes	no	0.45
	CBD	all	all			0.90	2%			yes	no	0.45
Collector	Urban	all	all			0.85	2%			yes	no	0.40

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Table 17 shows the computation of the capacities by facility type based upon the assumptions contained in Table 16. The results have been rounded off to the nearest 50 or 100 vehicles per hour per lane. The capacities per lane contained in this table would then be multiplied by the number of lanes (in one direction) at the critical point to obtain the critical point capacity for the facility.

Table 17
Example Computation of Default Capacities by Functional Class and Area/Terrain Type

Functional Class	Area Type	Terrain Type	Lanes	Ideal Cap	PHF	Fhv	Fw	Fdir	Fnopass	Fpark	Fleft	Fcbd	g/c	Cap/Lane
Freeway	Rural	Level	all	2400	0.85	0.98								2000
		Rolling	all	2400	0.85	0.91								1900
		Mountain	all	2300	0.85	0.80								1600
	Urban	all	all	2300	0.90	0.98								2000
Divided Arterial	Rural	Level	>2	2200	0.85	0.98								1800
		Rolling	>2	2100	0.85	0.91								1600
		Mountain	>2	2000	0.85	0.80								1400
	Suburb	all	all	1900	0.90	0.98				1.00	1.10	1.00	0.45	850
	Urban	all	all	1900	0.90	0.98				0.90	1.10	1.00	0.45	750
	CBD	all	all	1900	0.90	0.98				0.90	1.10	0.90	0.45	650
Undivided Arterial	Rural	Level	2	1400	0.85	0.95	1.00	0.97	1.00					1100
		Rolling	2	1400	0.85	0.83	1.00	0.97	0.93					900
		Mountain	2	1400	0.85	0.65	0.80	0.97	0.81					500
	Suburb	all	all	1900	0.90	0.98				1.00	1.00	1.00	0.45	750
	Urban	all	all	1900	0.90	0.98				0.90	1.00	1.00	0.45	700
	CBD	all	all	1900	0.90	0.98				0.90	1.00	0.90	0.45	600
Collector	Urban	all	all	1900	0.85	0.98				0.90	1.00	1.00	0.40	550

2.3.4 Evaluation of Capacity Techniques

The Highway Capacity Manual is the most generally accepted basis for computing highway capacity. However, it requires data not frequently available to planners. Consequently, Florida DOT and others have used defaults for some of the needed data to make the HCM method more useful for transportation modeling and planning applications. The Florida LOS Manual provides one set of defaults for applying the HCM method. NCHRP 3-55(2) provides a procedure for applying the HCM that allows the selective substitution of defaults for those data items not available in a particular locality.

Both the Florida and NCHRP 3-55(2) methods are poorly suited to estimating the capacity of arterials with no left turn bays and unprotected left turns. The HCM analytical process for this situation is difficult to approximate with a method suitable for planning purposes.

The method of substituting default values into the Highway Capacity Manual equations for estimating capacity has been used by planning agencies throughout the country. The FDOT method is used to develop capacities for the FSUTMS (Florida Statewide Urban Travel Modeling System). NCHRP 3-55(2) method is a newer variation of the FDOT method that has not yet been put into modeling practice.

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2.4 Potential Improved Speed-Flow Relationships

This section describes various methods that have been proposed in the literature for improving current planning techniques for estimating vehicle speeds as part of the travel demand forecasting process. The next chapter discusses more elaborate methods that can be used after the traffic assignment process is complete.

Most of the techniques use a single equation to estimate speed as a function of the free-flow speed and volume/capacity ratio. Two of the techniques incorporate the impacts of delays at nodes (intersections).

2.4.1 Horowitz Adaptation of HCM

Horowitz [14] developed a set of speed-flow equations and capacity and free-flow speed look-up tables based upon the 1985 Highway Capacity Manual. He also adds the capability to compute the impacts of intersection (node) delay on vehicle speeds.

Horowitz fitted three alternative functions (the BPR, Spiess (described in section 2.4.4), and Overgaard) to the HCM data for freeways and multi-lane highways and found that all three equations could fit the data quite well with the appropriate choice of equation parameters. The parameters however had to vary significantly between facility types and free-flow speeds. No single set of parameters could be used for all the possible free-flow speeds even within the same facility type. Table 18 illustrates the results for the BPR equation.

Table 18. Parameters of BPR Equation That Best Fit 1985 HCM

Facility	Free Speed	a	b
6-Lane Freeways	70 mph	0.88	9.8
	60 mph	0.83	5.5
	50 mph	0.56	3.6
Multi-Lane	70 mph	1.00	5.4
	60 mph	0.83	2.7
	50 mph	0.71	2.1

("a" is the recommended coefficient, "b" is the recommended power for the BPR equation)

If software limitations limit the user to only one equation with one set of parameters then Horowitz recommends that the BPR equation with parameters, a=0.83 and b=5.5, be used to predict link travel times. He recommends that the HCM procedures be used to determine link capacities for uninterrupted flow facilities (freeways, multi-lane highways) and that these procedures be applied on a link by link basis (rather than using a simple look-up table).

Horowitz recommends that an additional intersection or node delay be calculated for interrupted flow facilities which is then added to the estimated link travel time computed with the BPR equation. He suggests various modifications to the HCM stop sign and signal delay equations to make them more useful in a planning environment.

He explains the notion of "adaptive traffic control", which makes the computation of node delay more realistic for planning purposes. Traffic adaptive control means that as traffic volumes change in the future, the operating agency will naturally change the signal timing and even the type of intersection control (stop sign versus signal) in response to the new demand. Thus the capacity of an intersection is not fixed, but a function of the demand on all of the legs of an intersection.

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He recommends an approximate method for estimating intersection capacity for interrupted flow facilities:

$$cap = s \ [Y/(Y+Y^*)][C-L]/C \qquad \text{(eqn. 17)}$$

where:

cap = capacity of subject approach;

s = average saturation flow rate across all phases of the subject approach;

Y = the maximum of the flow ratios (volume/saturation) for the subject approach or the opposing approach;

Y^* = the maximum flow ratio (v/s) among all conflicting approaches (excludes the subject and opposing approaches);

C = cycle length (secs);

L = lost time for all phases in the cycle (secs).

A set of look-up tables of intersection capacities are provided for users wishing to avoid the above computation.

Horowitz [15] points out though that the incorporation of traffic adaptive control and the effects of crossing streams of traffic at an intersection results in multiple equilibria for the network. There may be multiple solutions to the traffic assignment equilibrium problem. Horowitz has demonstrated that multiple equilibria do exist on real world networks.

2.4.2 TMODEL Node Delay Method

TMODEL [16], a proprietary demand model development package, contains a node delay method for computing node delays based upon the entering volume, the conflicting volumes, node capacity, type of control, and the approaches being controlled. The node delay is then added to the link travel time to obtain the total travel time for the link.

The node delay is computed using the following equation:

$$d = k1(v/c+k2)^b + a \qquad \text{(eqn. 18)}$$

where:

d = node delay in minutes

k1 = parameter (ranges from 0 to 0.40)

k2 = parameter (ranges from 0 to 0.30)

b = parameter (ranges from 1 to 10)

a = zero volume delay (ranges from 0 to 0.20)

v/c = volume/capacity ratio.

Roadway nodes are classified into 8 classes: uncontrolled, freeway ramp merge, freeway ramp diverge, signalized, pedestrian signal, 2-way stop, 4-way stop, and yield control). Another 8 classes are used for

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other special nodes, and users can code up to 100 node classes. A separate set of parameters are provided for each node class according to whether the v/c ratio is under or above a certain limit.

For example, the following parameters are used for a signalized intersection (k1=0.25, k2=0.15, b=4, a=0.06) when the volume/capacity ratio is less than 0.85. The parameters change to: (k1=0.25, k2=0.15, b=10, a=0.06) when the v/c ratio exceeds 0.85.

Node capacity is computed using the following equation:

cap = k4*(entering capacity)   (eqn. 19)

where:

    cap   = approach capacity in vehicles per hour;

    k4   = capacity reduction parameter (equivalent to g/C ratio for signals)

    entering capacity = link or approach capacity before reduction for node limitations.

The capacity reduction factor ranges from 0.45 to 0.80 for signalized nodes.

2.4.3 Akcelik/Davidson Formula

Akcelik [17] proposed an equation derived from classical queuing theory for predicting the travel time on any road facility. The equation requires as input the free-flow travel time rate, the length of the analysis period, the capacity of the link and the travel time rate when the facility is at capacity. The equation predicts the inverse of speed, the travel time per unit distance.

$$t = t_0 + \left\{ 0.25T \left[(x-1) + \sqrt{(x-1)^2 + \frac{8J_A}{QT}x} \right] \right\}$$ (eqn. 20)

where:

t   = average travel time per unit distance (hours/mile)

t_0 = free-flow travel time per unit distance (hours/mile)

T   = The flow period, (typically one hour) (hours)

x   = the degree of saturation = volume/capacity

Q   = Capacity (vph)

J_A = The Delay Parameter

Akcelik's equation states that the travel time (t) is equal to the free-flow travel time (t_0) plus the average overflow queue (N_0) divided by the capacity (Q). The average overflow queue divided by capacity is the portion of the equation inside the brackets to the right of "t_0". The equation for the average overflow queue was estimated by Akcelik to take into account variations in queue lengths caused by random variations in arrivals.

The delay parameter J_A is a function of the number of delay causing elements in the section of road and the variability of the demand. Akcelik suggests lower values of J_A for freeways and coordinated signal systems. Higher values apply to secondary roads and isolated intersections.

The value of J_A can be computed if the difference in the rate of travel (hours per mile) between capacity and free flow conditions on the facility is known. Substituting x=1.00 in the above equation and solving for J_A yields:

$$J_A = \frac{2Q}{T}(t_c - t_0)^2$$ (eqn. 21)

where t_c = the rate of travel at capacity (hours per mile).

The equation explicitly takes into account the delays caused by queuing and can be applied to any facility type. The assumptions are that there is no queue at the start of the analysis period, and there is no peaking of demand within the analysis period (T).

Recent work by Dowling [18] suggests that the Akcelik curve can achieve accuracy superior to that of the standard BPR curve while also reducing the number of iterations required to reach equilibrium in the traffic assignment process. This is because the Akcelik curve becomes essentially linear at high v/c ratios.
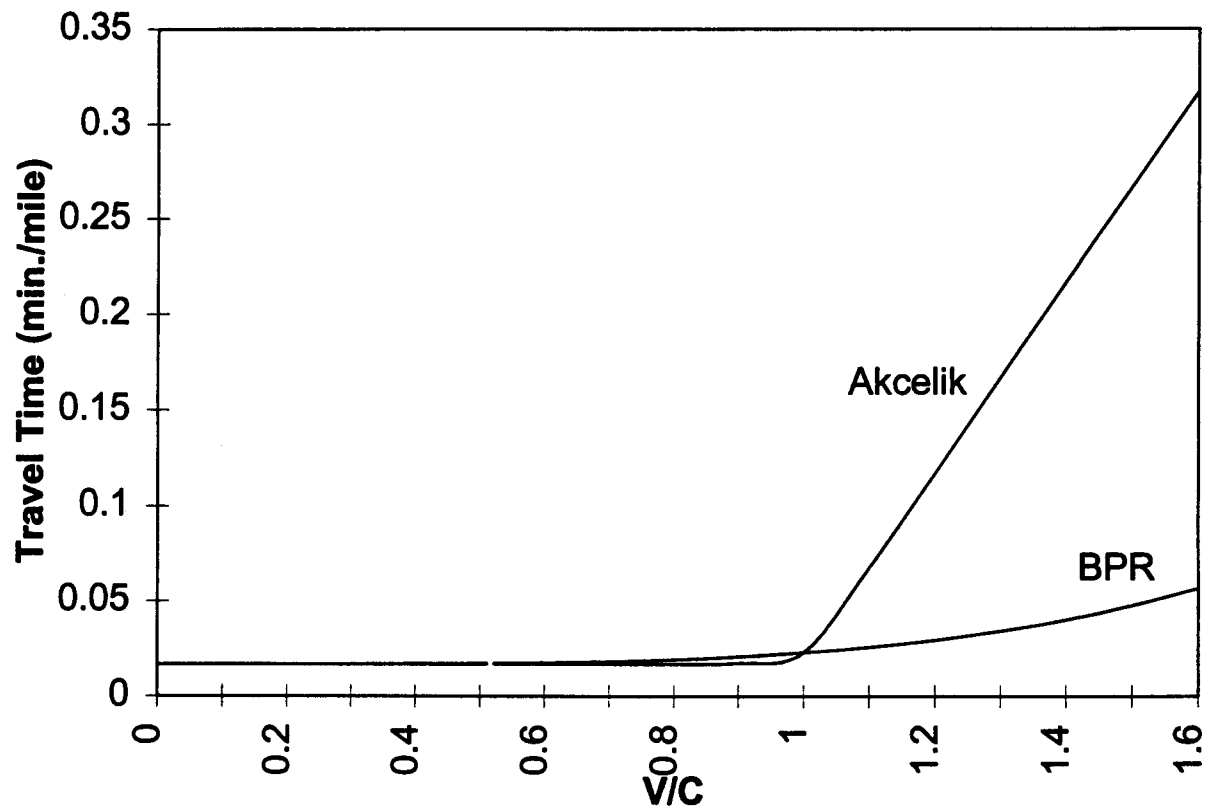

Figure 6. Plot of Akcelik and BPR Curves

2.4.4 Conical Delay Functions

Spiess [19] developed a revised speed-flow equation designed to enable computers to compute equilibrium traffic flows much more rapidly than with the standard BPR curve. From the perspective of computational

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efficiency, the BPR curve is highly volatile at high v/c ratios (a slight change in the forecasted volume results in large changes in the estimated speed) and is too insensitive at low v/c ratios (large changes in volumes result in minor changes in speed). The BPR curve also uses high power functions (exponents greater than 2) which slows computer computations. All of these characteristics of the BPR curve tend to slow down computer travel model computations of equilibrium traffic volumes.

Spiess suggests a "conical delay function" as a more computationally efficient speed-flow curve that still is very similar to the BPR curve. The conical delay function drops off fairly constantly over lower ranges of v/c ratios and does not increase as rapidly as the BPR curve at higher v/c ratio ranges. The equation is as follows:

$$t = t_0 * \left[\frac{t_c}{t_0} + \sqrt{a^2 * (1-x)^2 + b^2} - a*(1-x) - b \right]$$ (eqn. 22)

where:

t = travel time (sec)

t_0 = the travel time under free-flow conditions (sec)

t_c = the travel time at capacity (Spiess uses $t_c/t_0 = 2.0$)

a = a calibration parameter that must be greater than 1.

b = (2a -1) / (2a -2)

x = v/c ratio

Note: at capacity (x=1), the formula yields "$t = t_c$"; and at zero volume (x=0),the formula yields "$t = t_0$".

2.4.5 NCHRP 3-55(2) Updated BPR Curves

Dowling [20] recommended the following parameters for the BPR curve, based upon work by Skabardonis that fitted the BPR curve to various freeway and arterial data sets:

For Signalized Arterials: a= 0.05   b=10

For all other facilities: a=0.20   b=10

where "a" is the coefficient of the BPR curve and "b" is the exponent.

2.4.6 STEAM Model

Cambridge Systematics developed a speed model for use in its Surface Transportation Efficiency Analysis Model (STEAM) computer program (Cambridge[21]). The model predicts mean daily, peak and off-peak speeds given the ratio of average weekday daily traffic to the facility's hourly capacity. Separate models were fitted to freeways and to signalized arterials.

$$S = \frac{1}{\frac{1}{S_f} + D}$$

$$D = c_1 x^{c_2} \exp(c_3 x) \qquad \text{for x} <= c_0$$

$$D = c_4\left(1 - c_5 x^{c_6} \exp(c_7 x)\right) \quad \text{for } x > c_0$$

where:

S = Average speed in miles per hour

S_f = Free-flow speed in miles per hour. (Free flow speed is that which occurs when traffic volumes are very low. On interrupted flow facilities, they include delays due to traffic control devices but exclude any congestion related delays.)

D = Congestion delay in hours per vehicle-mile

x = Ratio of average daily weekday traffic to hourly capacity for the section (AWDT/Capacity)

c_0 to c_7 = Constants given by following table.

	Freeways			Signalized Arterials		
Constant	Daily	Peak	Off-Peak	Daily	Peak	Off-Peak
c_0	10.5	12.1	11.1	9.74	9.62	12.6
c_1	2.39E-08	2.35E-07	1.13E-07	5.62E-04	8.44E-04	4.35E-04
c_2	3.75	3.29	2.52	0.862	0.615	0.937
c_3	0.287	0.235	0.259	0.0739	0.124	0.0516
c_4	0.05	0.05	0.05	0.166	0.166	0.166
c_5	1.494E-02	2.865E-04	1.058E-03	1.313E-01	8.591E-03	1.177E-02
c_6	3.42	7.00	4.91	1.61	3.80	2.91
c_7	-0.372	-0.797	-0.449	-0.173	-0.407	-0.237

The model was developed by applying simulation models to several different facilities using different demand patterns obtained from field counts. The simulations took into account delays due to incidents, peak spreading observed in the field, day to day variations in demand, and decreases in capacity when demand exceeds capacity.

2.4.7 Evaluation of Improved Speed-Flow Curves

The standard BPR curve underestimates mean vehicle speed for flows below capacity and overestimates speeds for demands greater than capacity. It is insensitive to signal control parameters that are known to have as great an effect on vehicle speeds as volume/capacity ratio. The BPR curve started out consistent with the 1965 HCM, but it is now inconsistent with the 1985 and 1994 editions of the HCM.

Horowitz and Dowling both suggest changes in the parameters of the BPR curve that enhance its accuracy, make it more consistent with the current edition of the HCM, and allow the BPR equation to be used with actual capacity rather than practical capacity. The Horowitz equations were fitted to the 1985 Highway Capacity Manual data for freeways and multi-lane highways. The Dowling equations were fitted to 1994 Highway Capacity Manual data plus field data on signalized arterials.

Horowitz also adds the capability to estimate node delay using procedures that approximate the 1985 HCM method. Although, this improves consistency with the HCM, little is known about how the computation of node delay improves the accuracy of the speed estimates. Horowitz did find that the

incorporation of node delay in the traffic assignment process results in the presence of multiple equilibria. While this is a significant problem from a theoretical point of view, it is unclear how much of a problem it is from a practical application point of view. If the multiple equilibria are close to each other, then it matters little from a practical point of view, which equilibrium solution the traffic assignment process arrives at first.

TMODEL also incorporates a capability to estimate node delay. However, the theoretical basis for the node delay equations is unclear. Their impact on accuracy of the speed estimates is also unknown.

Spiess suggests a speed-flow equation that is superior to the BPR curve in terms of its computational speed. It has been adopted by Portland, Oregon for their metropolitan area model.

Akcelik proposes a speed-flow relationship based on time-dependent queuing with random arrivals. Limited testing by Dowling of Akcelik's equation suggests that this equation is superior to the standard BPR curve in its ability to replicate the speed estimates for "over capacity" conditions on freeways. The Akcelik equation however tended to over estimate delay for volumes ranging from 70% to 170% of capacity on arterials.

Figure 7 and Figure 8 illustrate the results. The test results shown in these two figures compare the various speed-flow curves to simulation model runs for the I-880 freeway in Hayward California, and Ventura Boulevard (a signalized arterial) in Los Angeles, California. The simulation models used were FREQ [22] model for freeways, and TRANSYT7F [23] for arterials.

A series of simulations were run at different demand levels. The demand levels were obtained by applying a uniform factor (eg. 50%) to the observed volumes on each facility. The average speed of through traffic over the entire length of the freeway or arterial and the maximum v/c ratio was computed for each demand level from the simulation results. The speed-flow curve speed estimates were computed using the maximum segment v/c ratio for each demand level.

Simulation models were used because they can estimate speeds for demand levels greater than capacity. Traffic counts could not be used for these tests because counts at a particular location are by definition limited to the capacity of the facility.

The Akcelik equation also appears to share some of the computational advantages of Spiess's equation by avoiding the use of higher power functions. A significant new burden to modelers when first using the Akcelik equation is the computation of the J_a parameter for each facility type and area type. Once this parameter is computed, the Akcelik equation is just as easy to apply as the BPR equation.

Comparison of Freeway Speed Prediction Techniques
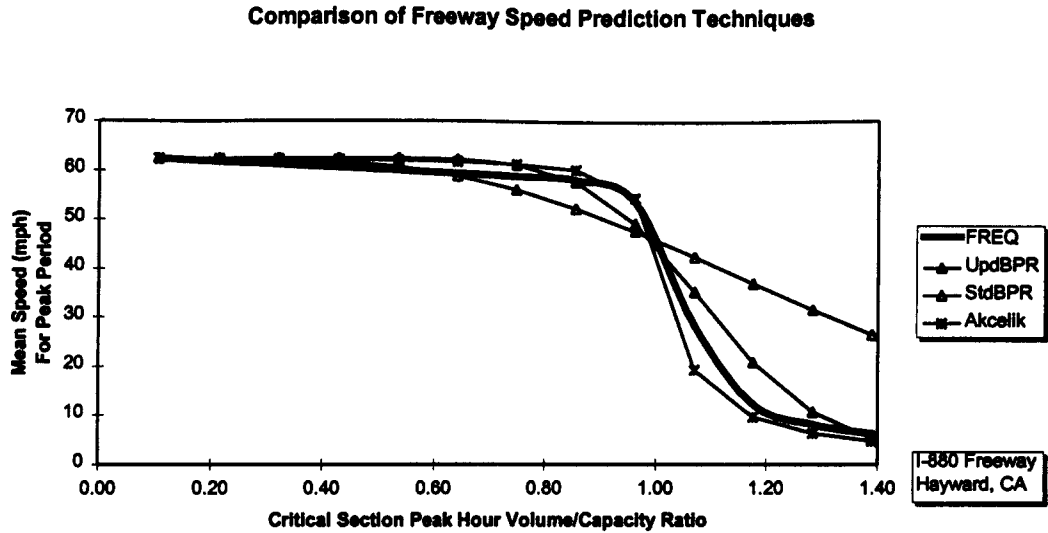
Figure 7. Performance of Freeway Speed Estimation Techniques

**Comparison of Arterial Speed Prediction Techniques
Over Simulated Range of Volumes**
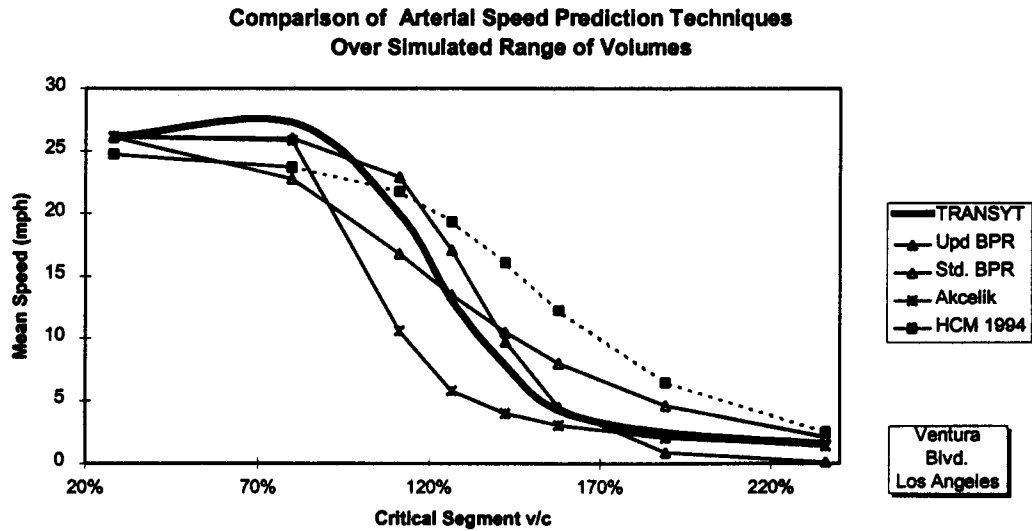
Figure 8. Performance of Arterial Speed Estimation Techniques[22]

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[22] Readers may note that the TRANSYT-7F estimated speed increases between 10% and 70% of the link capacity. This counter-intuitive result is due to the method used to select the range of volumes to be simulated. The method reduced or increased only the observed through volumes on the link. The number of left and right turns was held constant. Thus, at low through volumes, the turning volumes represent a large proportion of the traffic on the link and cause the average link speed (for all moves) to be slower than at higher through volume levels.

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2.5 Recommendations

It is recommended that planning agencies look into updating the parameters of their speed-flow curves to better reflect recent research on the impacts of volumes on freeway and arterial speeds. They may choose to use one of the forms of the BPR curve recommended by Horowitz, or Dowling. Alternatively they may choose to use the Akcelik equation which has the advantage of being based upon queuing theory and resulting in faster convergence to equilibrium than the BPR curve.

Modelers considering changes to their speed flow curves should recognize that the new curves require the use of actual capacity values for each link, not the lower planning capacities used in the standard BPR curve. Modelers may also wish to provide for peak spreading to allow for more accurate estimates of peak hour volumes. Otherwise the result of using these new speed flow curves may be to over estimate future congestion.

The accuracy of any speed-flow relationship hinges upon the quality of the input data used in the process.

Planning agencies should look into improving their ability to estimate link free-flow speeds. Planning agencies may choose to use the free-flow speed equations recommended by Dowling (which take into account the posted speed limit, signal spacing, and signal timing), or they may choose to use the NCHRP 3-45 and HCM free-flow speed equations (which are sensitive to geometric design parameters and will be contained in the 1997 Highway Capacity Manual).

Agencies should also look into their procedures for estimating link capacities. They may choose to use the Florida LOS Manual general table of service volumes or to develop their own estimates of maximum service volumes using the Florida table generating spreadsheets. Those agencies with more resources available and desiring greater sensitivity to geometric conditions may choose to use the NCHRP 3-55(2) formulae for estimating capacity.

Field surveys of free-flow speeds and capacities can significantly improve the speed estimation accuracy of travel demand models. Link specific free-flow speeds and capacities, rather than simple look-up tables of speeds and capacities by facility type and area type can significantly improve accuracy.

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Chapter 3. Assignment Post-Processors

This chapter deals with more elaborate procedures for estimating speed that cannot typically be incorporated in current travel demand model traffic assignment procedures either because of their more extensive data requirements or their impact on processing times.

Two post-processors are described that have been used in practice to refine the speed estimates produced by demand models. One technique was used in Boston on the Central Artery Project, the other technique is currently incorporated as an optional "pre-processor" to an air quality model used in California. The discussion concludes with a presentation of a post-processor method recommended as a result of the NCHRP 3-55(2) research.

3.1 Dowling & Skabardonis Method

One of the traditional problems with the incorporation of queuing analyses in transportation demand modeling has been the difficulty of tracking both the temporal duration and the geographical extent of the queue. Dowling & Skabardonis [24] demonstrated that reasonably accurate estimates of total system delay could be obtained by ignoring the geographical extent of the queues.

The method involves extending the peak hour demand forecast to a multi-hour peak period using locally available data on travel demand by hour of the day. Peak period demand is forecasted for each hour of the peak period based upon the peak hour forecast.

Average link speeds are then computed for each hour of the peak period using the hourly demands. If the demand during a particular hour exceeds the link capacity, the delay due to queuing is computed and added to the link travel time. Queues are carried over to the subsequent hour of the peak period.

The post processor was written as a macro in the MINUTP software package and tested against the FREQ and TRANSYT-7F traffic simulation models on a section of freeway and arterial street in Hayward, California.

All queues are stored on the link where the demand exceeds capacity. Queues are not propagated upstream, nor are they used to reduce downstream flows. The result of these simplifications is a series of over estimates and under estimates of the impacts of queuing that appeared to cancel out, at least under the limited testing performed by Dowling & Skabardonis.

3.2 NCHRP 255 Procedures

Pedersen and Samdahl [25] developed a recommended set of procedures for computing speed, delay, and queue length for freeways and arterials for under-capacity and over-capacity conditions. These procedures have not been written into software to our knowledge.

Their recommended procedures for under-capacity conditions are almost identical to the procedures contained in the 1985 Highway Capacity Manual for basic freeway sections and urban arterials. One difference is their procedure reduces the design speeds reported in the 1965 Highway Capacity Manual to average speeds using a formula developed by Makigami, Woodie and May [26], as follows:

AS = OS - [DS/10 * (1-v/c)] (eqn. 23)

where:

AS = Average Speed

OS = Operating Speed

DS = Design Speed

v/c = volume/capacity ratio

Operating speed is the "highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions..." (AASHTO [27]). Design speed, as defined by ASASHTO, is the "maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern". Operating speed must always be less than or equal to the design speed.

Their procedures for estimating average speed on freeways and arterials can be brought up to date simply by using the procedures contained in Chapters 3 and 11 (Basic Freeway Sections, and Urban and Suburban Arterials) of the 1994 HCM. There is no need to convert operating speed to average speed, since the new HCM reports average speed.

Pedersen and Samdahl, however recommend a pair of procedures that extend the Highway Capacity Manual methods to over-capacity conditions. These procedures were originally developed by Curry and Andersen [28]. One procedure uses "shock wave" analysis to predict queuing on freeways. The other procedure uses deterministic queuing to predict delay on interrupted flow facilities.

3.2.1 Freeway Shock Wave Analysis Procedure

This procedure uses the lower limb of the speed-flow curve for freeways that was reported in the 1985 HCM but is no longer included in the 1994 edition of the HCM.

The freeway is split into three subsections as shown in Figure 9. The first subsection is the bottleneck where the upstream demand exceeds capacity(often the section of freeway just downstream of an on-ramp). The second subsection is the queue immediately upstream from the bottleneck (often the section immediately upstream from an on-ramp). The third subsection is the remaining portion of the freeway upstream of the queue (this subsection may not exist if the queue extends the full length of the freeway study section). The freeway study section must be extended if the computations show that the queue extends upstream beyond the initially selected freeway study section.
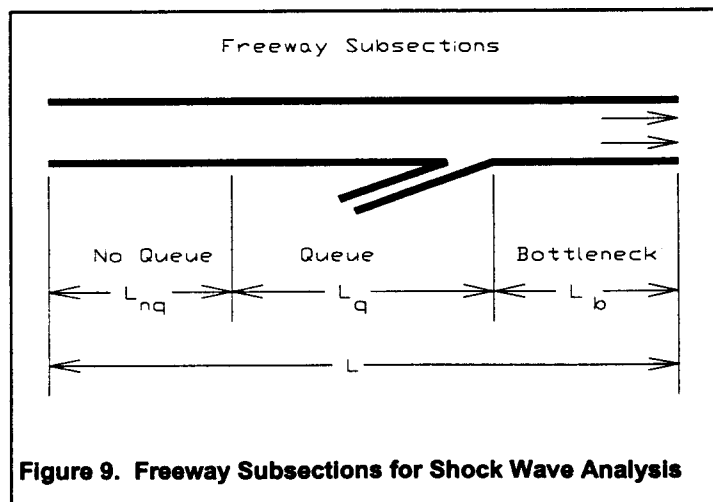

Figure 9. Freeway Subsections for Shock Wave Analysis

The average speed over the entire freeway section is then determined by averaging the speed in each subsection as shown in the following equation:

$$ARS = \frac{L}{\dfrac{L - L_b - L_q}{ARS_{nq}} + \dfrac{L_q}{ARS_q} + \dfrac{L_b}{ARS_b}}$$

<div align="right">(eqn. 24)</div>

where:

ARS	= Average Running Speed of entire freeway section
ARS_b	= Average running speed of bottleneck subsection of freeway
	= speed at capacity
ARS_q	= Average running speed in queue subsection upstream of bottleneck
ARS_{nq}	= Average running speed in subsection upstream of queue
L	= Length of entire freeway section
L_b	= Length of bottleneck section
L_q	= length of queue (see below for equation)

The bottleneck and non-queuing subsection speeds can be determined from the speed-flow curves shown in Chapter 3 of the HCM. The average speed within the queue section must be determined from the lower limb (the forced flow) portion of the speed-flow curve contained in the 1985 HCM.

The following equation, by Dowling, provides an approximate fit to the lower limb of this curve.

$$ARS_q = A * \exp[\ln B * (\tfrac{v}{c})^{1.27}]$$

<div align="right">(eqn. 25)</div>

where: A = 5, B = 6, v/c is the flow rate under queuing conditions.

This curve approaches 30 mph at v/c = 1.00, and 5 mph at v/c = 0.00. Parameters "A" and "B" can be modified according to the following equations if different speeds are desired:

A = the speed at v/c = 0.00

B = {the speed at v/c = 1.00} divided by "A"

The length of queue (L_q) is computed as follows:

L_q = {QR* T}/{2DQ}

<div align="right">(eqn. 26)</div>

where:

L_q	= the average queue length during the analysis period (miles)
QR	= the Queuing Rate (veh/hr)
	= Upstream Demand - Bottleneck Capacity
T	= Length of Time that the level of Demand occurs (Length of peak hour or peak period)(hrs)
	note that the queue is building and not dissipating during this period.
DQ	= change in vehicle density between queue and upstream non-queued subsection
	= {Bottleneck Capacity}/ARS_q - {Upstream Demand}/ARS_{nq}

3.2.2 Arterial Queuing Analysis for Over-Capacity

The average running speed for the arterial is computed using the same equation as contained in Chapter 11 of the HCM:

$$SPEED = \frac{[3600 * Length]}{[(RunningTimePerMile) * (Length) + D]}$$ (eqn. 27)

The difference is in the calculation of intersection delay (D) for those intersections on the arterial where the though movement volume/capacity ratio is greater than 1.00 (over congested intersections).

Step 1. Look-up the running speed for the link feeding the over congested intersection, the speed will be based on free-flow speed and signal density.

Step 2. Adjust the vehicle arrival rate for the fact that as the queue extends back from the intersection, vehicles join the queue "earlier" than they would have if the queue were at the intersection stop line.

$$AAR = Demand * \left\{ 1 + \frac{(Demand - Capacity)}{Lanes * Speed * 240 - Demand} \right\}$$ (eqn. 28)

Where:

AAR	= Adjusted Arrival Rate (veh/hr)
demand	= the predicted arrival rate of vehicles at the congested intersection stop line (veh/hr)
Capacity	= The saturation flow rate per lane times the number of through lanes times the green/cycle ratio for the approach (vphpl).
Lanes	= The number of through lanes on the approach (one direction)
Speed	= The average running speed for the approach found in step 1.
240	= The assumed queue density of 240 vehicles per lane per mile (22 feet per vehicle).

Step 3. Compute the Queue Length.

$$Q = 0.5 * \left\{ T * (AAR - Capacity) + Capacity * \frac{Cycle - Green}{3600} \right\}$$ (eqn. 29)

where:

Q	= the mean queue length (vehicles)
T	= Duration of Analysis period (hrs)
AAR	= Adjusted Arrival Rate (veh/hr)(from step 2)
Capacity	= maximum flow rate per lane times the number of lanes (veh/hr) (see Step 2).
Cycle	= the signal cycle length (sec)
Green	= effective green time for through vehicles (sec)

Step 4. Compute average Delay (D) at over congested intersection.

D = 3600 * Q/Capacity (eqn. 30)

where:

D   = average delay (sec)
Q   = mean queue length (veh)(from step 3)
Capacity   = saturation flow per lane * Lanes * Green/Cycle (veh/hr)

3.3 NCHRP 7-13 (Lomax) Curves

Lomax et.al.[29] used linear regression to fit a set of speed flow curves for arterials and freeways to various data sets they obtained as part of their research. The curves predict speed based on the volume/capacity ratio, signal spacing, and frequency of access points.

For Freeways:

$$Speed = 91.4 - 0.002[ADT / Lane] - 2.85[APM]$$ (eqn. 31)

where:

Speed = Mean peak hour speed (mph)

ADT = average daily traffic.

APM = access points per mile.

For all Arterials:

$$Speed = \frac{60}{\frac{60}{S_f} \cdot (1 + SD)^{0.3} \cdot \left(1 + \left(\frac{v}{c}\right)^4\right)^{0.7}}$$ (eqn. 32)

where:

Speed = Mean Peak Hour Speed (mph)

S_f = Free Flow Speed (mph)

SD = Effective Signal Density which is equal to:

= (1 - bandwidth/cycle) x (signals per mile)

v/c = peak hour volume to capacity ratio.

Bandwidth is the number of seconds during which a vehicle arriving at the first signal on the arterial can expect to traverse all of the signals of the arterial without stopping. Cycle is the cycle length in seconds for the arterial.

3.4 Margiotta Formulae

Richard Margiotta, et. al. [30] used the TRAF family of traffic simulation models to develop quadratic and exponential regression equations for predicting mean facility speeds as a function of the ratio of the average daily traffic (ADT) to the hourly capacity of the facility. The functions predict the delay due to traffic flow and the density of traffic signals per mile. The delay is added to the free-flow travel time to obtain total travel time.

The following set of equations was developed for freeways and multi-lane rural highways:

If x<=8, then:
$$d = 0.0611x + 0.00777x^2$$
(eqn. 33)

If 8 < x <= 12, then:
$$d = 28.4 - 7.16x + 0.467x^2$$
(eqn. 34)

If x > 12, then:
$$d = -31.7 + 2.98x + 0.0393x^2$$
(eqn. 35)

where:

x = the ratio of ADT to hourly capacity,

d = the ratio of hours delay to 1000 vehicle miles traveled.

The following equations were developed for urban arterials with signals and left turn bays (they were unsatisfied with the ability of the simulation models at that time to simulate delays for intersections without turn bays).

For n <= 20 and
x <=7:
$$d = (1 - \exp(-n/24.4))*(68.6 + 16.9x)$$
(eqn. 36)

For n <= 20 and
7 < x <= 18:
$$d = (1 - \exp(-n/24.4))*(186.9 + 14.6(x - 7) - 1.85(x - 7)^2) + 0.706(x - 7)^2$$
(eqn. 37)

where:

n = the number of signals per mile

x = the ratio of ADT over hourly capacity,

d = the ratio of hours delay per 1000 vehicle miles traveled.

Similar equations were also developed for urban arterials without traffic signals and rural two-lane roads.

The delay equations are quick and simple to apply and ideal for the estimation of speeds for the Highway Performance Monitoring System (HPMS). These equations unfortunately are heuristic approximations of simulation model results from artificial data sets. Their application is therefore limited to the particular facility types and conditions on which the equations were developed. They cannot be relied upon when signal timing, signal coordination, and demand peaking characteristics vary from the simulated data sets used to develop the equations.

3.5 The HPMS Analytical Process

The Highway Performance Monitoring System Analytical Process [31] provides a process for estimating link speeds as a function of an initial running speed plus various adjustments for pavement conditions,

curves, grades, speed change cycles, stop cycles, and idle time. The initial running speed is determined from a look-up table based on the facility type and the congestion level.

The speed adjustments are applied in sequence, first the initial running speed is reduced according to pavement conditions, which is then further reduced for the effect of curves, etc.. The speed adjustment for curves is applied only if the safe speed on the curve is lower than the reduced speed based on pavement conditions. The speed adjustment for grades is applied only to trucks. The adjustment for speed change, stop cycles, and idle time is a function of facility type and volume/capacity ratio.

3.6 Ruiter Adaptation of HCM

Ruiter [32] demonstrated how the analysis procedures contained in the 1985 Highway Capacity Manual (HCM) could be used to develop facility specific speed-flow relationships through the use of pre-selected default values for various input items required by the HCM. Default values for various HCM input items are selected based on facility type, facility type subgroup, and area type. Ruiter then shows how the substitution of the default values results in one or more simplified equations that can be used to predict link speed. Ruiter illustrates the development of a simple equation combined with a look-up table for use in predicting freeway speeds. He also illustrates the development of a set of equations for computing signalized arterial speeds. None of these equations can be generalized, since they depend on the specific default values selected, however; the equation development procedure can be applied to any situation where the HCM techniques can be applied.

Ruiter suggests two equations for extending the HCM speed predictions to conditions where demand exceeds capacity.

For freeways and expressways:

$$S_p = S_{p1} * (0.555 + 0.444*(V/C)^{-3})$$ (eqn. 38)

where: S_{p1} = speed at v/c = 1.0

For arterials and collectors:

$$S_p = S_{p1.2} * (0.663 + 0.583 *(V/C)^{-3})$$ (eqn. 39)

where: $S_{p1.2}$ = speed at v/c = 1.2

These equations were developed for use in the Phoenix metropolitan area [33]. Ruiter recommends that peak spreading be applied to the demand volumes to reduce the over prediction of delay for high demand volumes that would result with these equations.

3.7 Boston Central Artery Post Processor

Bechtel/Parsons Brinkerhoff and Cambridge Systematics [34] developed and applied a post-processor process that adjusted the forecasted link volumes and speeds output by the TRANPLAN software package for the Boston Central Artery Project. The various highway links in the model network were first grouped into five link types (see previous chapter discussion on the types). The demand model forecasts were reviewed and revised to correct for any volume calibration errors observed in the base year model run. The corrected volumes were then input into the specially developed speed-flow equations derived from the Highway Capacity Manual and queuing theory (see previous chapter discussion on the speed formulae). The revised volumes and speed estimates were then output to a TRANPLAN readable file which was then read back into TRANPLAN.

The travel time prediction equations were developed for the following link types:

Type 1 Links: Links where the travel time is constrained by signalization,

Type 2 Links: Links where the travel time is constrained by geometrics,

Type 3 Links: Expressway and ramp links with v/c < 0.7678,

Type 4 Links: Expressway and ramp links with v/c > 0.7678, and

Type 5 Links: Links where the times are unconstrained.

Link types 1 and 2 use the same travel time formula but with different default values for some of the parameters.

Note that the third term in the equation is a deterministic queue delay formula for conditions when the v/c is greater than 1.00.

$$T = T_0 + \frac{0.38 * C * (1 - \frac{g}{c})^2 * PF}{(1 - \frac{g}{c}X)}$$
$$+ 173X^2 \left[(X-1) + \sqrt{(X-1)^2 + 16\frac{X}{cap}} \right] * PF$$
$$+ 1800 * \left(\frac{V}{cap} - X \right)$$

(eqn. 40)

where:

T = Congested Travel Time (seconds)

T_0 = Free-flow travel time (seconds)

C = cycle length (seconds)

g = green time (seconds)

PF = Progression adjustment factor (set to 1.0)

X = Minimum of volume/capacity ratio or 1.00

V = Demand volume (vehicles per hour)

cap = Capacity (vehicles per hour)

The travel speed on Type 3 links is determined by computing the volume/capacity ratio and looking up the speed in the Highway Capacity Manual. When the v/c ratio reaches 0.7678, then the formula for link type 4 is used to compute the speed.
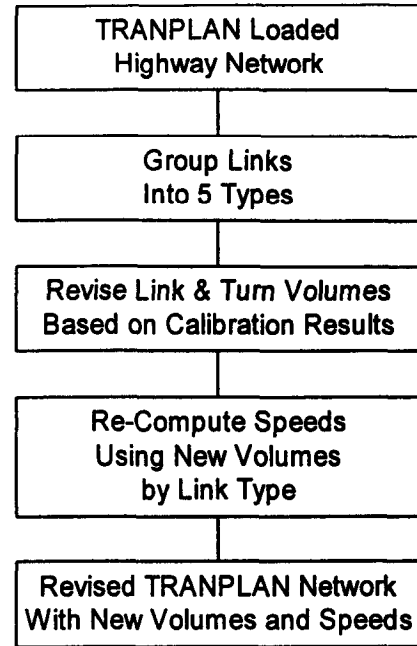
Figure 10. Flow Chart for Boston Central Artery Post Processor

TRANPLAN Loaded Highway Network

Group Links Into 5 Types

Revise Link & Turn Volumes Based on Calibration Results

Re-Compute Speeds Using New Volumes by Link Type

Revised TRANPLAN Network With New Volumes and Speeds

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The travel speed on link types 4 and 5 is computed using the BPR formula with an "a" coefficient of 0.15 and a "b" power of 6.

3.8 DTIM2 Speed Post Processor

SAI created a computer program for Caltrans, called the Direct Travel Impact Model (DTIM2) [35], that reads the loaded highway network produced by transportation planning software (TRANPLAN, MINUTP, and EMME2), and computes the corresponding pollutant emissions by 2 km grid cells within the region. The DTIM model contains an optional speed post-processor developed by Dowling [36] that uses 1985 Highway Capacity Manual techniques and queuing analysis to compute more accurate estimates of link speeds by hour of the day, over a 24 hour period.

The DTIM2 speed processor contains a set of speed-flow curves and equations for signalized and unsignalized facilities. These curves and equations have been verified on California freeways, rural highways and signalized arterials.

Congested speeds on unsignalized facilities are estimated using variations of the BPR curve fitted to the speed-flow curves contained in Chapter 3 of the 1994 Highway Capacity Manual (HCM).

The DTIM data collection effort showed that rural highways have speed-flow curves similar to freeways when adjusting for the different free-flow speeds. Thus only a single set of speed-flow curves are provided for freeways and unsignalized highways.

Congested speeds on signalized facilities are estimated using the 1994 HCM procedure for signalized arterials. This procedure estimates speeds and signal delay based on signal spacing, capacity, and signal timing. The processor provides all of the needed signal timing and signal spacing data according to the facility type and area type of each highway link. The user can also directly input this signal data for specific links. The user can also edit the file of default signal data by area type and facility type to suit the conditions specific to the study area.

Figure 11. Speed Processor Flow Chart
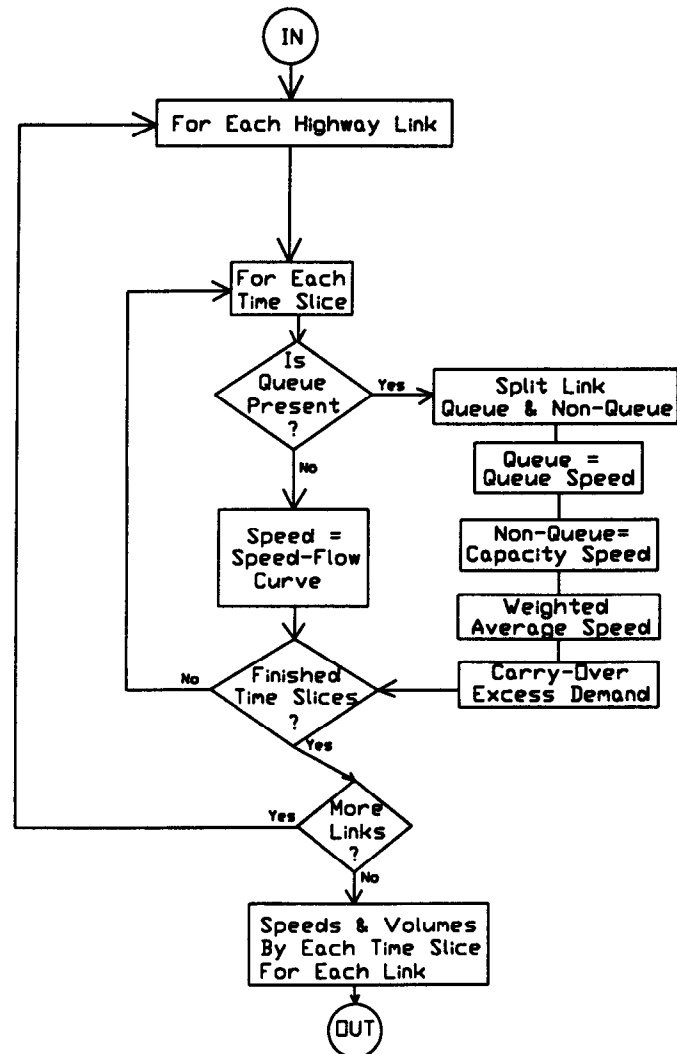
The HCM procedures contained in the DTIM2 speed processor are valid only for volumes less than capacity. The speed processor thus also contains a queuing analysis algorithm for use when volumes exceed capacity. The queuing algorithm splits the day into one hour long time slices. The total demand is allocated to each time slice according to peaking factors provided by the user.

Experience to date with the DTIM2 speed processor has found that it estimates speeds significantly lower than models using traditional BPR curves. This makes it difficult for planning agencies to switch to the speed processor because of inconsistency problems with previous forecast work by the agency.

3.9 NCHRP 3-55(2) Procedure

NCHRP 3-55(2) recommended a post processor procedure that estimates the space mean speed and level of service for one direction of a facility over the entire peak period. The analysis takes into account delays due to signal control and to queuing. This procedure has not yet been implemented in software.

The recommended procedure varies according to whether the study facility is signal controlled or not.

Unsignalized Facilities

The recommended procedure for unsignalized facilities is based on the analysis procedures contained in Chapters 3, 7, and 8 of the 1994 Highway Capacity Manual. The facility is divided into subsections (within which demand and capacity are relatively constant). The traffic demand in the peak period (if more than one hour long) is divided into a sequence of hourly demand rates. A simplified HCM analysis is then applied to each segment for each hour of the peak period. Excess demand in one hour on one segment is carried over to the following hour (but the queue is not propagated to upstream segments in order to save on computational complexity).

The capacity and delay impacts of ramp merge, diverge points and weaving sections are neglected in this procedure.

The hourly capacity of each segment in one direction is determined using the capacity equations by facility type contained in this section. The objective is to fill in a table of capacities by segment and hour.

The next step is to check whether the demand on any segment exceeds its capacity for any hour within the peak period. If so, then the excess demand must be carried over to the following hour and the queue delay computed for the current hour.

If a queue is determined to exist, then the queuing delay (due to demand exceeding capacity) is computed using the following equation.

$$d_q = 3600 * T * \left(\frac{V_{t-1} + V_t}{2c} - 1 \right)$$
(eqn. 41)

where:

d_q	= Mean delay due to excess demand (sec).
T	= Duration of time period (hrs)
3600	= Converts hours to seconds.
V_{t-1}	= Leftover demand from previous time period (t-1).
V_t	= Additional demand occurring in current time period (t).
c	= Capacity of segment in subject direction (veh/hr)

The segment running times are computed for each segment (i) and time period (t) using the following equation:

$$R_{i,t} = 3600 * \frac{(1 + a(\frac{v}{c})_{i,t}^{b})}{S_f}$$ (eqn. 42)

where:

$R_{i,t}$ = Mean segment running time per unit length for segment "I" and time period "t" (sec/mi, sec/km)

S_f = Mean segment free flow speed (mph or kph)

$v/c_{i,t}$ = Ratio of volume to capacity for the segment

a = 0.20

b = 10

The space mean speed over the entire peak period and the total study section length of a freeway, multi-lane-highway, or two-lane rural road is estimated using the following equation. Delays due to demand exceeding capacity on any one segment are added to the individual segment travel times, which are then summed over the entire study section to obtain the total travel time over the length of the study section. The total travel time is then divided into the total study section length to obtain the space mean speed for the study section.

$$s = \frac{3600 * N_t * \sum_i L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{i,t} dq_{i,t}}$$ (eqn. 43)

where:

s = Space mean speed over the length of the facility (mph or kph).

L_i = Length of segment "I" (mph or kph).

$R_{i,t}$ = Running time for segment "I" during time period "t" (sec/mile or sec/km).

$dq_{i,t}$ = Delay due to queuing on segment "I" and time period "t"(sec).

N_t = Number of time periods being analyzed.

Procedure for Signalized Facilities

The NCHRP 3-55(2) recommended speed estimation procedure for signalized facilities requires the estimation of signal timing for the facility. The g/C ratio (green time per cycle) for the through movement and the cycle length © must be estimated for each intersection of the facility. The queue overflow and queue delay at the intersections are then computed.

The running time is computed based upon the mid-block free-flow speed, which is in turn computed based upon the posted speed limit. The node delay for signalized intersections is computed using equations adapted from Chapter 11 of the Highway Capacity Manual.

The space mean speed in one direction over the length of a signalized facility and over an entire analysis period is computed using the following equation:

$$SPEED = \frac{3600 * N_t * \sum_i L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{j,t} dn_{j,t} + \sum_{j,t} dq_{j,t}}$$ (eqn. 44)

where:

Speed = Space mean speed over the length of the facility (mph or kph).

N_t = Number of time periods (t) within analysis period.

L_i = Length of segment "I" (mph or kph).

$R_{i,t}$ = Running time for segment "I" (sec/mile or sec/km).

$dn_{j,t}$ = Delay at node "j" for through traffic in the subject direction during time "t".

$dq_{j,t}$ = Delay due to demand exceeding capacity at node "j" during time period "t".

Dowling compared the speeds estimated using the NCHRP 3-55(2) post-processor process against field data (see Figure 12) and found that the post-processor (labeled "Enhanced ARTPLAN" in the figure) is superior to non-post processor methods (Standard BPR, and Enhanced BPR in the figure) but not as accurate as using the HCM directly to estimate speeds.

Figure 12. Results of NCHRP 3-55(2) Post-Processor Evaluation
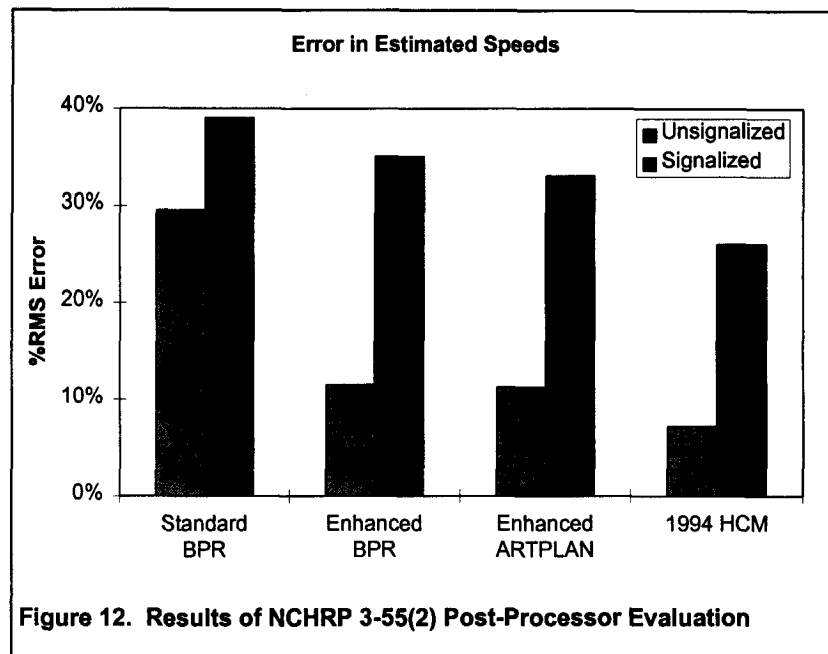
3.10 Evaluation of Post Processors

Most of the post-processors described above attempt to apply the analytical methods contained in the Highway Capacity Manual to the estimation of capacity and speed in travel demand models. Default values are substituted for the more difficult input items. Figures in the HCM are converted to look-up tables. Iterative steps in the HCM procedures are either dropped altogether or replaced with a simplified approach.

The Highway Capacity Manual however does not treat situations where demand exceeds capacity so many of the post-processor methods also include a method for computing the delays due to queuing.

A few post processor methods (Margiotta, and Lomax for example) completely avoid the HCM (and its data requirements). They fit simple linear or curvilinear speed estimation equations to real world or simulated data.

The Dowling & Skabardonis method computes queues on individual links without worrying about propagating their effects to upstream or downstream links. The method extends peak hour analysis to an entire peak period. The method is easily automated and can be applied to the entire model highway network.

NCHRP 255 presents a complete post processor method for applying the HCM method and queuing analysis to specific facilities on the network. This method does not address multi-hour analyses, but can easily be extended to peak period analyses by repeating the analysis steps for each hour and carrying the excess demand over to the next hour. The procedure is not designed to deal with multiple queues that may interfere with each other.

Both the Margiotta and the Lomax speed-flow equations predict the impact of daily traffic and signal density on speed, but they are linear or non-linear equations fitted by regression techniques to either real or simulated data. One must be very cautious in applying these equations outside of their calibration range.

The HPMS method is oriented to facility specific analyses. The initial running speed is selected from a look-up table (by facility type, speed limit, congestion level, development type, number of lanes, etc.). This initial speed is then modified based upon grade, curves, and speed change cycles on the selected facility. This method is also difficult to update since it relies upon an extensive set of look-up tables and charts that would need to be regenerated each time the HCM is updated.

Ruiter suggests a useful approach for developing link specific speed-flow relationships based upon the Highway Capacity Manual. The link specific geometric and signal control values are substituted into the HCM procedures and solved for a specific speed-flow equation for that facility. Look-up tables are used for uninterrupted flow facilities which do not have explicit speed-flow equations in the manual. He also extends the HCM method to over-capacity conditions by adding queuing delay equations.

Look-up tables are not hard to include in software; if the categories, variables and dimensions are known in advance; but it is difficult to include in the software the flexibility for users to change the tables every time a new edition of the HCM is published (Current expectations are that 1997 and year 2000 editions of the HCM will be published). Also, it is somewhat laborious to solve the HCM equations for many different facility types with a range of input variables.

The Boston Central Artery method includes a manual check and revision of the volume forecasts before applying HCM based look-up tables and equations for estimating speed. The method extends the HCM to queuing situations as well. The speed estimation steps of the method have been automated and can be applied to the entire network. The method does not directly address multi-hour analyses.

The DTIM2 method applies HCM techniques to estimating speed along with a multi-hour queuing analysis procedure. User specified or default peaking factors are used to estimate hourly demand within the peak period. The method allows planners to specify default input data (such as average signal spacing and cycle length) by facility type and area type. The method is automated and can be applied to the entire highway network. Experience to date in California indicates that this method produces significantly lower speed estimates than traditional BPR curve based approaches. Caltrans is currently investigating the addition of a peak spreading capability.

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The NCHRP 3-55(2) post processor method is oriented toward the analysis of a single facility composed of perhaps a dozen links. The overall average through travel speed for the facility is estimated for the entire peak period. The method splits the peak period into time slices of one hour each, and splits the facility into a series of analysis segments. Queuing is identified and carried over to the following time slice. Queues however are not propagated upstream, nor are downstream demands reduced due to upstream bottlenecks.

3.11 Recommendations

If the planning agency has the necessary data resources and desires greater accuracy in the speed forecasts (without affecting the calibration of the travel demand model) then an assignment post-processor to improve the estimated vehicle speed may be appropriate. The limited data available to date however indicates that the overall gain in accuracy (over an improved and updated BPR curve) may not be proportional to the amount of additional data and effort required (see Figure 12).

The addition of queuing analyses to the speed estimation process will enhance accuracy but runs the risk of over-estimating congestion by ignoring the impact of congestion on peak spreading. Ruiter suggested that both queuing and peak spreading should be implemented in tandem. Caltrans is currently investigating this issue.

The major advantage of post processors is that they allow the planner to test the impacts of facility design and operation options that can not be tested in a traditional travel demand model. The sensitivity of the system performance to signal control and road geometry can be tested by means of a post processor.

Post processors also extend the ability of planners to analyze traffic operations over the length of the peak period, rather than being limited to 24 hour or a peak hour analysis. The DTIM software, the NCHRP 255 method, and the NCHRP 3-55(2) method provide techniques for evaluating speeds over the length of the peak period.

Chapter 4. Prediction of Trips by Vehicle Operating Mode

As the Clean Air Act Amendments of 1990 have been implemented by the EPA, the interface between network-based travel demand models and air quality emissions models have become more important. Vehicle activity data taken from the network travel model are being used as inputs to the emissions estimate. The air quality analysis procedures combine vehicular travel data with emission rates from an emission factor program. Emission factors are provided by vehicle type and region of the country and are dependent upon the ambient temperature, vehicle speed, vehicle volumes, operating mode, time-of-day, and year of analysis. Travel demand models have been used to derive many of these variables. Since the initial intent of the travel demand models was not for air quality analysis, several model processes and post-processors have been developed to disaggregate travel demand data.

This chapter focuses on the prediction of trips by vehicle operating mode. A major component of the mobile source emissions analyses is the proportion of trips that are made in the cold start mode, which is the period before the engine and/or catalytic converter is completely warmed up. Vehicles in the cold start mode emit higher concentrations of total organic gases (TOG), carbon dioxide (CO), and nitric oxides (NOx). How this increased emissions is addressed as part of the emissions inventory currently depends heavily upon which emission factor model is used. Since the EPA requires the use of MOBILE5 in all states except California, much of the following discussion centers around the MOBILE5 defaults for operating mode fractions used to estimate the proportion of vehicle-miles-traveled (VMT) made in the cold start mode. Alternative mode fractions are discussed and compared to the MOBILE5 defaults. Other approaches are also discussed.

The chapter includes a definition of operating modes, a description of current emission factor models, a discussion of current regional models and post-processors, an evaluation of the current approaches, and recommendations for improving the emission estimates from vehicles in the start mode. Issues for further study that could not be addressed as part of this study are also identified.

4.1 Definitions

Motor vehicle emissions can be separated into exhaust emissions and evaporative emissions. Emissions due to evaporation occur both when the engine is on and off. Evaporative emissions include running evaporative, hot soak (stop), diurnal, and resting losses. Exhaust emissions are associated with the emissions from the engine when the vehicle is in operation. The exhaust emissions can be separated by operating mode into transient (start) emissions and running emissions. Running, or hot stabilized, emissions occur when the engine and catalyst are at normal operating temperature.

Emission levels vary widely by vehicle operating mode. When the vehicle is first started, the exhaust emissions are greater than when the engine is warmed up and operating in a stabilized condition. Start emission are higher because of the low air to fuel ratios and poor performance of cold catalytic converters. Starts are associated with higher concentrations of carbon monoxide (CO) and hydrocarbons (HC) or Total Organic Gases (TOG). Exhaust emissions of nitric oxides (NO_x) are not as sensitive to the cold engine. TOG emissions show the greatest increase when the vehicle is operating in cold transient mode.

The Federal Test Procedure (FTP) driving cycles are used to define the operating mode and to estimate the emission rates. The FTP is a dynamometer test that simulates actual highway driving. Emissions are collected for each of the three cycles (bags) shown in Table 19. Each bag of the FTP cycle has a time duration, distance, and average speed associated with it. The cold transient emissions were measured upon engine start-up after the vehicle has sat without starting for a period of from 12 to 36 hours. After

505 seconds, the engine was assumed to be operating in hot stabilized mode. The hot transient emissions was measured upon an engine start-up after the vehicle sits for a ten minute period during the FTP, after the stabilized running portion.

Table 19. Definition of FTP Cycle				
FTP Cycle	**Operating Mode**	**Duration**	**Distance**	**Avg. Speed**
Bag 1	cold transient	505 secs.	3.58 miles	25.6 mph
Bag 2	hot stabilized	867 secs.	3.85 miles	16 mph
Bag 3	hot transient	505 secs.	3.58 miles	25.6 mph

Since emission rates vary depending on the extent to which the engine and catalyst have reached their optimal operating temperature, the start emission rates vary with the vehicle soak time, which is the time that the engine has been turned off. For the emissions modeling, the start emission varies depending upon the duration of the soak and the vehicle type. EPA has defined a cold start for non-catalyst vehicles as a start after four hours of non-operation and for a catalyst vehicle as a start after one hour of non-operation.

A distinction needs to be made between the cold or hot start modes which are associated with the trip origin and the operating modes - cold transient, hot transient and hot stabilized - which occur during the actual trip. Each of the emission factor models approach start emissions differently. MOBILE requires the fraction of VMT in the hot and cold transient modes, while EMFAC calculates hot and cold start emissions factors that are associated with the engine start at the trip origin.

As an indirect means of estimating cold start emissions, the travel time or distance from a trip origin is used as an indicator of operating mode. The duration of the transient modes (before the engine has attained the hot stabilized operating mode) is specified by the FTP as the first 505 seconds or 3.58 miles after the start.

The FTP was developed in the 1970s and there is some question as to the applicability of the FTP to current driving behavior. The introduction of the catalytic converter has had a significant effect on mobile source emissions. The changes in technology and driver behavior raise the need to revisit the FTP drive cycle and the definition of operating modes. Recent studies seem to indicate a shorter duration for vehicles operating in the cold start mode. However, for the purposes of this study, the FTP definitions of operating modes will be used.

4.2 Emission Factor Models

Emission factor models produce emission rates by pollutant that are specific to the vehicle type, temperature, vehicle fleet, and calendar year for a particular geographic region. For all states, except California, the use of emission rates from MOBILE is required by the EPA for air quality analyses. In California, where the air quality standards are higher, the California Air Resources Board (CARB) has developed the Emission Factor (EMFAC) model for estimating emission factors for use in the air quality analysis. Each of these models are described below.

4.2.1 MOBILE

MOBILE was developed by the EPA to estimate emission rates. MOBILE produces a composite emission rate for each pollutant. The base emission rates for light-duty vehicles and trucks are developed from testing over the FTP (and in MOBILE5, from IM240 tests and the correlation of IM240 and FTP test results[23]). In order to estimate in-use emission factors for conditions other than the standard FTP, which are temperatures of 75°F, a diurnal temperature range of 60°-84°F, an average speed of 19.6 mph, and fuel volatility of 9.0 psi RVP (Reid Vapor Pressure), correction factors are applied to the basic emission rate estimates. Emission rates for trip start, trip end, and diurnal emissions are based on vehicle-miles-traveled (VMT). MOBILE separates evaporative emissions by components, but does not separately account for cold and hot starts. The increased emissions from starts is included as part of the exhaust emissions total which is measured in grams of pollutant emission per mile.

A required variable in the Scenario input section of MOBILE5 is operating mode fractions. The operating mode fraction is a percentage of the total VMT. The default values for the operating mode fractions are:

PCCN	Cold start	Non-catalytic	20.6%
PCHC	Hot start	Catalytic	27.3%
PCCC	Cold start	Catalytic	20.6%

These values are based on the FTP driving cycle and can be used to derive the stabilized catalytic and non-catalytic and the hot start non-catalytic fractions. These percentages are areawide averages. The guidance provided in the User's Guide for Mobile5 states "in the absence of supporting data for values other than those listed above, EPA believes that the values reflecting FTP conditions are appropriate in many cases."[37] Although EPA acknowledges that more representative operating mode fractions may be developed for modeling localized conditions or limited time periods as well as areas with average trip lengths that are significantly shorter or longer than 7.5 miles.

Future updates to MOBILE may include changes to this approach for modeling start emissions. When a preliminary list of potential improvements for incorporation into MOBILE6 was ranked by the Modeling Working Group, several of the improvements that were considered high priority were related to start emissions. The list included reevaluation of warm/cold start assumptions and separation of trip start emissions and running emissions[24]. [38] Changes to the current approach used by MOBILE5 to model start emissions would impact how the travel demand models interface with the emission factor models to forecast emissions.

4.2.2 EMFAC7F

The California Air Resources Board (CARB) has developed the Motor Vehicle Emission Inventory (MVEI) series of models for estimating mobile source emissions. One component of the MVEI emission estimate models is the Emission Factor (EMFAC) model. The EMFAC model uses base emission rates and activity weighting and mileages to produce fleet composite emissions factors. An emission factor is calculated for each process, pollutant, vehicle class, technology and model year.

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[23] IM240 is a vehicle test cycle for the enhanced inspection and maintenance program which consists of a portion of the Federal Test Procedure.

[24] MOBILE5 includes start emissions as part of total exhaust emissions which is calculated using VMT. Travel demand models are used to forecast VMT for the emissions calculation. If start emissions are modeled separately, the travel demand models can be used to estimate starts and operating modes by link.

EMFAC produces composite emissions rates for exhaust and the evaporative emissions. Start and running emission rates vary by temperature and dew point. Each of the processes modeled in EMFAC are shown in Table 20. The pollutant units and the vehicle activity basis used for the emissions calculation are also shown in the table.

Table 20. EMFAC7F Emission Model		
Process	**Pollutant Units**	**Vehicle Activity Basis**
start-up	g/trip	vehicle trip ends
stabilized running exhaust	g/mile	VMT by speed
hot soak evaporative	g/trip	vehicle trip ends
diurnal evaporative	g/veh/hr	population of vehicles
resting loss	g/veh/hr	population of vehicles
evaporative running losses	g/mile	VMT

The EMFAC7F model [39] assumes an incremental start emissions factor, where running emissions are assumed for the entire trip with an increment added to account for the increased emissions during cold and hot starts. The start emission rate is calculated as the difference between the FTP bag 1 (bag 3) and the speed-corrected bag 2 emission rate. It is defined on a per trip basis, assuming the FTP trip length of 3.58 miles. The speeds and trip lengths used to derive the start emissions rate are defined by the FTP cycle.

EMFAC7F defined a "cold start" as a period of engine off of at least one hour for catalyst equipped vehicles and 4 hours for non-catalyst equipped vehicles. Hot starts are defined as a period of engine off for less than one hour for catalyst equipped vehicles and less than four hours for a non-catalyst equipped vehicle.

4.2.3 EMFAC7G

A new start emissions methodology was incorporated in the latest version, EMFAC7G [40]. The methodology allows for variable soak times rather than limiting it to hot and cold starts. A different test cycle – Unified Cycle – was used to estimate the cold start emission factors. A new soak activity distribution divided start emissions into twelve (12) categories rather than the two – hot and cold.

For the start-up emissions, the emissions are based on the vehicle starts. The start-up emission rate is divided among cold starts and starts after variable hot soak periods. The cold start emission rate is derived from the FTP bag 1 - cold start by adjusting the emission rate to the rate from the first 100 seconds of a specially designed cycle. This adjustment considers the first 100 seconds of a cold start as the significant portion of a cold start.

In addition to the cold start emission rate which is based on a 12-hour soak period, start soak fractions were developed from test vehicles with soak periods ranging from 0 to 12 hours. The soak fractions are applied to the cold start emission rate for each of the various soak periods. Currently, the same soak distributions are used for all calendar years and all counties at this time.

With EMFAC7G, emissions are estimated for starts rather than for trips. The distribution of starts by vehicle age are based on the U.S. EPA Instrumented Vehicle Study and Caltrans survey data. The California statewide weighted average number of starts/vehicle/day is 6.35 for Light-Duty Automobiles, which is higher than the average number of trips per vehicle per day that was based on survey data. The difference can be attributed non-destinational trips, i.e. short side trips, shuffling cars in driveways, and moving cars in a lot, that are not typically included in the travel surveys.

4.3 Current Models for Mobile Source Emissions Modeling

The current approach to estimating emissions is to combine vehicular travel data with emission rates from an emission factor model. The emission factor models provide emission rates by pollutant for a range of vehicle types. The emission inventories are dependent upon the ambient temperature, vehicle speed, vehicle volumes, type of highway facility, operating mode, time-of-day, and year of analysis. Travel demand models have been used to derive many of these variables. Since the initial intent of the travel demand models was not for air quality analysis, several processors have been developed to disaggregate travel demand data.

4.3.1 Travel Demand Models

Travel demand models are network-based vehicle activity models that follow the four-step transportation modeling process – trip generation, trip distribution, mode choice, and trip assignment – to forecast travel on the network. Typically, these models provide daily estimates of travel on the highway network. These models were initially developed to forecast the future capacity needs of the transportation system.

Several procedures have been developed within the travel demand models that allow the direct estimation of trips that are made in the cold start mode. These procedures are largely bookkeeping procedures where the initial portions of the trip that are made in the cold start mode for a specified time duration are assigned to links and retained as a separate link variable. The vehicle volumes by cold start and normal running mode can be used to derive the operating mode fractions for use in place of the MOBILE5 operating mode fractions. The procedures for tracking cold starts are described below for the EMME2[41]and MINUTP[42] model software packages. TRANPLAN has the capabilities to track cold start vehicles, but the option is not currently available as part of the standard software package.[43]

The capability of travel demand models to predict trips by operating mode on a link-by-link basis has the potential to improve the emissions estimates by better accounting for increased start emissions. The models can potentially be used to provide estimates of operating mode fractions that are specific to the roadway network, time-of-day, and trip purpose.

4.3.1.1 EMME2

The EMME2 model allows the user to track cold starts on the network as part of an additional optional auto assignment. A special cutoff operator allows the user to specify what portion of the path generated by the assignment is to be considered. The cutoff operator allows the computation of the volumes corresponding to the first X kilometers(or miles) or minutes of a car trip., in order to use these volumes for detailed emissions modeling. The user can specify the duration of the start mode by elapsed distance or travel time. The assignment results are the auto volumes that correspond to the cutoff specified. The assignment also generates a matrix containing the vehicle-miles corresponding to the specified cut-off for each O-D pair. The user must factor the trip table matrices by start mode.

4.3.1.2 MINUTP

In MINUTP, starts are tracked by factoring the trip table matrices into hot, warm, and cold starts and by specifying a time before the vehicles reach the stabilized operating mode. The "COLDSTART" parameter under assignment allows the user to specify the distance or time the trips from a particular trip matrix must travel before leaving the start/transient operating mode. The user must specify the matrices to which these apply. The trip table matrices are factored by an estimate of the percentage of trips making cold or warm starts. This trip table is then assigned to the network using the "COLDSTART" parameter. The user can specify outputs as start volumes by link, percent relative to total link volumes, or both.

4.3.2 Emissions Inventory Models

Several models have been developed to prepare emission inventories using the emission rates from the emission factor models and vehicle activity from the travel demand models. Some models provide direct interface with transportation models, while others allow for transportation model data to be used as input. Some models do not require the use of transportation model data at all.

Several models have been developed by states and local air districts to perform their emissions inventories. Of the emission inventory models currently available, there are two approaches. Since EPA requires the use of the MOBILE emission factors, most of the emission inventory models interface with MOBILE5. These emissions models base their emissions estimates on VMT since MOBILE provides only composite emission rates that combine start emissions with the running emissions. Trip end emissions are not tracked separately. With the exception of California, alternative emissions estimation procedures have not been approved by EPA. This, in effect, has limited the accounting of start emissions to adjustments to the operating mode fractions in MOBILE. The difference among the MOBILE-based emission inventory models is the level of detail required to run the model and the difference in the user-friendliness. In California, the emissions inventory models utilize the EMFAC emission rates and the transportation activity data to perform emissions inventories. These emissions inventory models require vehicle activity data that separates trip starts and stops. These models tend to be more data intensive.

4.3.2.1 BURDEN

The BURDEN[44] model is the fourth of the MVEI series developed by CARB for estimating emissions inventories. The BURDEN model combines the emission factors with area-specific data to produce emission inventories. Estimates in tons of pollutant per day by county, air basin, and statewide.

As inputs to BURDEN, vehicle activity data from the California Department of Transportation (Caltrans) or the Department of Motor Vehicles are primarily used. Larger regions of the state use VMT estimates from their transportation models for some classes of vehicles. The vehicle activity data includes VMT, vehicle population, trips taken (hot/cold starts), speed distributions, soak distributions, and temperature profiles. The BURDEN model output includes emission inventories by county/air basin. BURDEN splits activity data into six time periods. Using emission rates from EMFAC, BURDEN calculates running, start, and evaporative emissions for each of the six time periods. The percentage of starts for each soak time ranging from 1 minute to 720 minutes is provided by Caltrans and the U.S. EPA for each county/air basin.

Vehicle starts are estimated from the per vehicle start rate and the fleet population. The per vehicle start rate for light-duty and medium duty vehicles are estimated from data from the U.S. EPA's Instrumented Vehicle Study combined with Caltrans Survey Data. For regions with travel demand models, adjustments are made to the total number of starts according to the total number of trips predicted by the models. Travel demand models are also used to estimate the VMT by speeds. The CARB staff analyzed EPA

study data to produce soak distributions at 12 soak times for each of the six time periods. These soak distributions are used for all calendar years, for all counties.

4.3.2.2 DTIM/PC-DTIM

DTIM was developed for use with EMFAC and provides gridded outputs for Urban Airshed Modeling (UAM). DTIM was developed to read trip assignment output from Caltrans statewide travel demand model and emission rates from EMFAC to produce emissions estimates in gridded format. Start and evaporative emissions are calculated from the number of trip origins and destination within a traffic analysis zone. The running emissions are calculated from the link vehicle-hours-traveled multiplied by the hourly emission rate. DTIM produces better spatial and temporal resolution of emissions since the cold start and hot soak emissions are derived from trip ends, not VMT and assigned to a particular grid cell based on the location of the traffic analysis zone. Similarly, PC-DTIM, the PC-based version, calculates link level, hourly emissions using vehicle activity data from regional travel demand models and emission rates from EMFAC.

One of the inputs to the DTIM model is a file containing starts, parks, and stables. The starts include percent trip starts by hour of day and the percent of cold starts by vehicle technology type by hour of day.

4.3.2.3 IMPACT

The IMPACT[45] program was developed by Texas Transportation Institute (TTI) for use with MOBILE5. The program produces both regional and gridded emissions estimates. The program divides emissions into start-up, hot soak, hot running, and diurnal emissions. Start-up emissions are applied at the trip origin, stop (hot soak) emissions are applied at the trip destination. Hot running emissions are distributed along the path of the trip. IMPACT projects emissions on a vehicle trip basis rather than the VMT basis typical of MOBILE models. IMPACT can be used to estimate emissions for an entire urban area or along a major transportation corridor over a ten to twenty year period. The program is not used in Texas since it has not been approved by EPA.

4.3.2.4 Other Emissions Models

The state-of-the-practice report prepared by the Texas Transportation Institute (TTI) [46] included several other models in their review of emissions models. Most of these models use MOBILE5 emission rates as input and therefore, do not address trip start emissions separately from the composite emissions rate. These models, including MoVEM, PERF, and PPAQ, apply trip start, trip end and diurnal emissions on a VMT basis. It is likely that when trip end emission rates become available, these emissions models will be modified to use trip end emission rates.

4.3.3 Data Sources

Data for estimating vehicle operating modes can be separated into operating mode fractions and start percentages. The operating mode fractions are percentages of VMT and are associated with the vehicle trip. MOBILE5 uses these operating mode fractions to calculate composite emission factors for each of the pollutants. The start percentages apply to trip starts and is the portion of trip starts that can be categorized as cold, warm or hot starts or by varying soak periods.

While the travel demand models can be used to estimate the link-by-link cold start percentages, the user must define the cold starts as a time or distance impedance and the percentage of cold and hot starts for each traffic analysis zone. While the time or distance can be defined using the FTP definition of cold start, the percentages of starts for each traffic analysis zone has been shown to differ by area, time-of-day, and trip purpose.

4.3.3.1 Operating Mode Fractions

Researchers and practitioners generally feel the need to develop operating mode fractions for varying situations. Several studies have been conducted that stratify mode fractions and compare the results to the standard FTP fractions. Approaches include cross-classification of survey data and the use of network-based travel demand models to estimate operating mode fractions.

The National Personal Transportation Survey (NPTS) is a periodic survey on personal travel. The 1990 NPTS is based in travel diaries for households throughout the United States and includes trip information such as trip purpose, mode, trip length, day-of-week, time-of-day, vehicle used and vehicle occupancy. Several studies have used the NPTS data to estimate the operating mode fractions and VMT by operating mode. Data from the National Personal Transportation Survey (NPTS) were utilized to distinguish between cold transient and hot transient modes.

Venigalla et. al. (47) used NPTS data to derive vehicle operating modes by trip purpose and by time-of-day. Using the identification of the vehicle, the start time, and length of each trip, a start mode and operating mode were associated with each trip end. It was assumed that all 1975 and later vehicles were equipped with catalytic converters and only 25 percent of those vehicles prior to 1975 were equipped with a catalytic converter. Based on their assumptions, only about 5 percent of the vehicles in the NPTS database were identified as catalyst-equipped. Therefore, the results of the study were derived for non-catalyst-equipped vehicles only.

Operating mode fractions were analyzed by trip purpose, time-of-day, and size of urban area. The following tables summarize the resulting operating mode fractions of VMT. The percent VMT for each operating mode was stratified by trip purpose and hour of day.

Table 21 shows the operating mode fractions by trip purpose that were derived from the NPTS. The percent of VMT in the cold transient mode ranges from 25.2 percent for non-home based trips to 35.4 percent for home-based work trips. For home-based work trips, the mode fractions vary significantly from the mode fractions for all trips. Given the increased emissions from vehicles in the cold transient mode, these differences by trip purpose could have a marked impact on the regional emissions estimates. When stratified by time-of-day, the percent of VMT in cold transient mode was higher during the night hours and the early morning hours, with the peak occurring during the morning commute hours between 6 and 9 am.

Table 21. Operating Mode Fractions by Trip Purpose			
	Cold Transient	Hot Transient	Hot Stabilized
Home-Based Work	35.4	7.4	57.3
Home-Based Other	30.0	22.8	47.2
Non Home-Based	25.2	27.2	47.6
All Trips	31.2	18.7	50.1

Table 22 shows the operating mode fractions by size of the urban area that was derived from the NPTS. These results proved to be less conclusive than with trip purpose. While the results showed variation in

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start mode fractions by the size of the urban area, they do not indicate a clear trend. In the larger urban areas, the percent of VMT in the stabilized operating mode tends to be higher.

Table 22. Operating Mode Fractions by Size of Urban Area			
Urban Area Population	Cold Transient	Hot Transient	Hot Stabilized
50,000 - 199,999	32.9	21.9	45.2
200,000 - 499,999	35.9	23.7	40.4
500,000 - 999,999	31.0	18.6	50.4
Over 1,000,000 (no subway)	29.8	18.1	52.1
Over 1,000,000 (with subway)	30.8	18.8	50.5
Nationwide	31.2	18.7	50.1

4.3.3.2 Percent Starts

National and localized travel behavior surveys can also provide information to estimate start mode fractions at the trip origin. Local survey data was assembled from areas in Alabama and for Boston as part of studies conducted in the late 1970s. The San Diego Association of Governments (SanDAG) used data from a 1986 Travel Behavior Survey to estimate the percent of daily trip ends starting up for their emission inventory.

Venigalla et al. also derived start mode fractions of trips from the NPTS data. [48] The start mode percentages could be used as direct inputs to EMFAC7F or as a means of deriving operating mode fractions of VMT for MOBILE5. Start mode fractions were derived from the NPTS database by trip purpose and by time-of-day. The results of the analysis are summarized in the following tables.

Table 23 shows the average daily start mode percentages by trip purpose. Start percentages differ dramatically by trip purpose. As might be expected, about four out of five of the home-based work trips start in cold mode. More non-home based trips are hot starts than cold starts.

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Table 23. Start Modes at Trip Origins by Trip Purpose		
	Cold Starts	Hot Starts
Home-Based Work	79.1	20.9
Home-Based Other	53.4	46.6
Non-Home Based	43.0	57.0
All Trips	57.1	42.9

The study stratified start mode percentages by time-of-day. The variation over each of the one hour periods was significant, ranging from about 50 percent cold starts during the midday to about 90 percent during the early commute hours. Table 24 summarizes the time-of-day start mode percentages into four periods roughly corresponding to the AM and PM peak hours, midday and night. Cold starts are highest during the AM peak hours when most vehicles have been sitting overnight.

Table 24. Start Modes at Trip Origins by Time-of-day		
	Cold Starts	Hot Starts
6 am to 10 am	69.1	30.9
10 am to 3 pm	51.8	48.2
3 pm to 7 pm	53.8	46.2
7 pm to 6 am	59.4	40.6
24-Hour Period	57.1	42.9

The results of the study indicate that based on the NPTS, trip purpose was found to be the most important variable affecting cold start mode percentages. Time-of-day also had an effect on cold start percentages. When cross-classified by trip purpose and time-of-day, the results indicate that for home-based work trips, the share of cold starts are highest during the morning hours and tends to decrease over the course of the day.

4.3.3.3 EPA Instrumented Vehicle Study

The U.S. EPA conducted vehicle instrumentation studies in Baltimore, Spokane, and Atlanta. The studies involved installation of activity recording devices in randomly selected vehicles being tested at vehicle inspection facilities. Activity data was collected on a second-by-second basis. The data included an estimate of starts per vehicle per day. The average number of starts per day tended to be higher than the trips per day. In California, the statewide starts per vehicle per day was estimated to be 6.35, which is higher than the 3.76 trips per vehicle per day that was previously used for emissions estimates. For

regional emissions estimates, a factor was applied for each air basin to trips per vehicle per day to estimate the starts per vehicle per day.

4.4 Evaluation

Unlike the post-processors and model improvements described in the earlier chapters, the need for predicting vehicles by operating mode is tied directly to the emission factor model required by the U.S. EPA. For the 49 states that are required to use the emission factors from MOBILE5, the emissions from vehicles in the start modes are calculated by multiplying the VMT by the composite emissions rates, which is based on the vehicle mode fractions of VMT and are not directly calculated from the number of starts. For California, the EMFAC model produces emission rates by trip starts and variable soak times as well as vehicle travel.

While some travel demand models offer the capability to track operating mode on a link-by-link basis, the emission factor models and the post-processors currently available would need to be modified to allow for the calculation on a link basis.

4.4.1 Comparison between MOBILE and EMFAC

Since the operating mode fractions have been shown to differ by time-of-day and roadway class, the areawide average that MOBILE applies may not present the most accurate estimate of start emissions. The reliance on average operating speeds and VMT to estimate emissions is overly simplistic and does not separately account for start and end emissions and the number of vehicles. The approach used in EMFAC begins to directly account for the difference between start and running emissions. However, since the methodology of each emission factor model differs, a direct comparison of the emission rates produced by each of the model is not possible. The emission rates from each model are not directly comparable because MOBILE calculates a composite emission rate that includes adjustment to the Base Emission Rate for operating mode, while EMFAC provides separate emission rates for start and running modes.

The approach using MOBILE allocates start emissions uniformly across the region being modeled based on VMT, while EMFAC produces a start emission rate that is allocated to the location of the start. The current approaches do not spatially allocate the start emission along the links where they occur. EMFAC tends to provide a better spatial-temporal resolution of emissions than MOBILE since the start and stop emissions are modeled separately from the running emissions. The default operating mode fractions used in MOBILE5 for estimating cold starts may not account for the variations in the highway network and traffic analysis zone structure. Since EMFAC treats start and soak emissions as proportional to trip ends (start and stops) rather than as an adjustment to a composite emission rate that is proportional to VMT, the spatial allocation of start emissions would appear to be more representative of where the start emissions are concentrated.

4.4.2 Mode Fractions from Network Models

A study conducted by Chatterjee et. al.[49] using data from the Sacramento area found variations in operating mode fractions by facility type and location. The methodology used network and trip data from the regional model and trip start data. Operating mode fractions were developed for five time periods – morning, afternoon, midday, night, and all day. One of the inputs to the process was the proportions of hot and cold starts by trip purpose and hour of day. This information was based on NPTS data. The Traffic Assignment Program for Emission Studies (TAPES) was developed for determining operating mode fractions on network links. The model assignment outputs include the total volume, the cold

transient volume, the hot transient volume, and the hot stabilized volume on each link. The FTP 505 seconds was assumed as the threshold between transient and stabilized operating modes.

Researchers applied a technique for tracing the elapsed time of vehicles from trip origins during the traffic assignment of zone to zone trips on a highway network and determine the proportions of transient and stabilized operating modes on every link of the network. The final results include operating mode fractions stratified by functional facility type and geographic location relative to the central business district as shown in Table 25 and Table 26.

The results of the model data showed that operating mode fractions varied substantially by location. Table 25 shows the operating mode fractions by location that was derived from the model. In rural areas, the overwhelming majority of the VMT (80%) was operating in the hot stabilized mode, which is almost double that of urbanized areas. For the urbanized areas, including the central business district (CBD), the CBD fringe, and outlying business districts, the operating mode fractions of VMT did not differ significantly. Suburban areas exhibited mode fractions similar to the urbanized areas.

Table 25. Operating Mode Fractions by Location			
	Cold Transient	Hot Stabilized	Hot Transient
CBD	34.93	42.31	22.76
Fringe	34.46	43.45	22.09
Outlying Business Districts	36.97	39.52	23.51
Suburban	30.86	48.95	20.19
Rural	11.40	79.84	8.76
All Areas	25.82	56.98	17.20

When stratified by facility type, the percent VMT in the cold transient mode is lower for higher classes of roads. Table 26 shows the operating mode fractions by facility type that was derived from the model. As might be expected, the model results showed that about 75 percent of the VMT on freeways is operating in the stabilized mode. Whereas on local roads, the percent of VMT in cold transient mode was the highest.

Table 26. Operating Mode Fractions by Facility Type

	Cold Transient	Hot Stabilized	Hot Transient
Freeway	14.73	75.10	10.17
Expressway	27.29	54.99	17.72
Major Arterial	34.85	41.37	23.78
Minor Arterial	34.02	44.07	21.91
Collector	39.60	35.40	25.00
Local Roads	43.29	29.13	27.58
All Roads	25.82	56.98	17.20

Table 27 shows the operating mode fractions by time-of-day that were derived from the model results. The operation mode fractions are shown to differ during the course of the day. During the night and early morning hours, the VMT percentage in the cold transient mode is highest and tends to decrease as the day progresses.

Table 27. Operating Mode Fractions by Time-of-Day

	6 am - 10 am	10 am - 5 pm	5 pm - 9 pm	9 pm - 6 am	24-hours
Cold Transient	29.7	24.20	23.60	34.91	25.82
Hot Stabilized	59.57	56.80	57.37	56.14	56.98
Hot Transient	10.73	19.00	19.03	8.95	17.20

4.4.3 Comparison of Mode Fractions

For the evaluation, the default FTP operating mode fractions from MOBILE5 were used as the source of comparison. A comparison of operating mode fractions is shown in Table 28. Both the survey data (NPTS) and the model data (TAPES) show a significant difference from the FTP operating mode fractions. The FTP appears to under-estimate the cold transient percentages and over-estimate the hot transient percentages. The percentage of VMT in the hot stabilized mode is fairly consistent between the FTP and the NPTS data, while the model data shows a slight increase compared to the FTP. The effects of these differences are even more significant when one considers the increased emissions of vehicles in the cold transient mode. Both the NPTS and the model data appear to indicate that the FTP hot transient fraction is two to almost three times higher than that derived from the NPTS and the model data. Some of

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these differences in mode fractions from the NPTS data may be attributed to the reduced number of non-catalyst vehicles that was assumed when analyzing the NPTS data.

Table 28. Comparison to FTP Operating Mode Fractions					
		NPTS		TAPES Model	
	FTP	7-9am	Daily	6-10am	Daily
Cold Transient	20.6	34.5	31.2	29.70	25.82
Hot Transient	27.3	13.7	18.7	10.73	17.20
Hot Stabilized	52.1	51.8	50.1	59.57	56.98

The operating mode fractions from the TAPES model are specific to the particular network and the assumptions used for the start percentages. The values may not be applicable elsewhere. However, the process used to derive these mode fractions could be replicated using a regional travel demand model. When compared to the FTP operating modes, the model derived operating modes show about a 50 percent increase in VMT fraction for vehicles operating in the cold transient mode during the AM peak period.

4.5 Recommendations

Improvements to the emissions inventory modeling processes can be made to improve the ability to estimate emissions from vehicles in the start transient modes of operation. These improvements involve changes to operating mode fractions that are input to the MOBILE5 emission factor model as well as improvements and modifications to travel demand models to disaggregate the travel activity data. The following recommendations include short-term travel demand model improvement as well as long-term improvements to the emission modeling process as a whole. Since the current MOBILE5 model requires operating mode fractions, many of the short-term improvements are directed at improving the mode fractions.

With the variability of mode fractions by trip purpose, time-of-day, size of the urban area, facility type, and locations, one approach to improving the start emissions estimates is to modify the mode fractions used by MOBILE5. Improvements to the current operating mode fractions could include:

1. Adjust the overall mode fractions to the mode fractions derived from the NPTS database.

2. Adjust the overall fractions using mode fractions derived from the network models.

3. Disaggregate the mode fractions and travel activity, then sum emissions for a daily average.

However, these approaches do not provide an accurate spatial dispersion of emissions since it allocates the start emissions based on the VMT. The mode fractions do not provide the spatial-temporal distribution that becomes more important for CO emissions. Another approach is that used by CARB which separates start emissions from running emissions and provides a better spatial distribution of start emissions. The estimates of cold and hot start percentages are key to this approach. Any modifications to the start mode percentages would affect the emissions estimate. A long-term improvement is to utilize

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the network model's capability to track cold starts by link. These recommended approaches are described below.

4.5.1 Alternative Operating Mode Fractions

From the NPTS survey data, the difference in operating modes compared to the FTP values and the variability of operating mode fractions by trip purpose and time-of-day warrant changes to the default operating mode fractions from MOBILE5. Alternative operating mode fractions could be applied for all trips or at some level of disaggregation. A simple approach is to use the operating mode fractions that were derived by Venigalla et. al. from the NPTS database. The following approaches use the travel demand model to derive the operating mode fractions.

For the 49 states that are required to use MOBILE5, the travel demand models can be used to predict the operating mode fractions specific to that network coverage area. Several improvements can be made to the current output from travel demand models to provide disaggregate data for the emissions inventory. Some of these improvements can be made internally or as a post-processor after the network assignment. By disaggregating the data from the travel demand model, the intent is to isolate those factors that affect the operating mode fractions. The degree to which the data can be disaggregated will depend upon the data that is available.

4.5.1.1 Overall Mode Fractions

The travel demand models can be used to estimate the mode fractions that are used by MOBILE to derive the composite emission rates. Software programs such as EMME2 and MINUTP allow the user to track trips that are made in the cold transient mode as they travel through the network. The user needs to provide an estimate of the percent of trips on the cold and hot start modes and the duration of the cold or hot transient trip. Assuming the FTP cycle for operating modes, the 505 seconds/3.58 miles can be used as the cut-off for transient operation. The data from the surveys and instrumented studies can be used to derive the percentage of starts by zone. Based in the NPTS database, cold and hot starts account for 57.1 and 42.9 percent of all starts, respectively. For greater detail, start percentages can also be divided by trip purpose and time-of-day. As examples, Table 23 provides estimates of start percentages by trip purpose, while Table 24 provides these percentages by time periods.

The model can provide link traffic volumes by operating mode. By summing the link VMT by operating mode, operating mode fractions of VMT can be calculated that are specific to the area being analyzed. These mode fractions can be input to MOBILE5 in place of the FTP defaults.

4.5.1.2 Disaggregate Mode Fractions

The travel models could be used to derive operating mode fractions by trip purpose, time-of-day, or other variables that have been shown to affect operating mode fractions. Start percentages, such as those shown in Table 23 and Table 24, can be used to derive operating mode fractions of VMT by trip purpose and by time-of-day.

The review of operating mode fractions found trip purpose to be one of the most important factors in predicting vehicle start modes. By providing estimates of travel by trip purpose, the emission estimate can be derived from emission factors developed from corresponding operating mode fractions. The travel demand models typically aggregate trip tables for each trip purposes into one trip table for assignment to the roadway network. By allowing the user to track trips by purpose during assignment, the model could be used to calculate VMT by trip purpose. Emission rates by trip purpose could calculated:

1. directly from operating mode fractions derived from the NPTS database as shown in Table 21 or some other survey data, or

2. from model-derived operating mode fractions by trip purpose, if the model was also capable of tracking trips by operating mode as well as by purpose.

These emission factors by trip purpose could then be applied to the VMT estimates by the trip purpose. The regional emission estimate would be a sum of emissions estimate by trip purpose.

Based on survey data, the operating mode fractions have been shown to differ by time-of-day. In order to reflect the time-of-day difference in the emissions estimate, the travel activity data would need to be provided with a similar time-of-day breakdown. Many of the travel demand models currently provide vehicle activity estimates on a daily basis, although some have the capability to provide estimates of travel during the peak periods. With the differences in cold starts by time-of-day, the need to disaggregate this data becomes even more important. This can occur either as a post-processing of daily trip assignments or through revising the model to directly produce trip assignments on a peak period basis. The time-of-day distributions of travel and the directional distribution of travel could be derived from national or local survey data. By applying emission rates that are specific to the time period, emission estimates can be made by time period and then summed for an average daily emissions estimate.

The operating mode fractions by time-of-day can be derived directly from the model by tracking cold starts through the assignment process and calculating overall percent of VMT for each operating mode, if the network model is capable of peak period assignments and tracking starts. Otherwise, operating mode fractions by time-of-day can be derived from the NPTS survey database or from other local surveys. By isolating emissions by time periods, in particular, the morning commute period, the emission estimate will better account for periods with disproportionate levels of vehicles in the cold transient mode when emissions are highest.

This approach of alternative operating mode fractions can be taken even further by disaggregating mode fractions by more than one variable. Operating mode fractions can be cross-classified by trip purpose and time-of-day. The resulting emission rates would be specific to that particular trip purpose during the specified time period. The same approach could be applied by facility type, location or other link variables that can be used to disaggregate travel activity.

4.5.2 Alternative Start Mode Percentages

To provide a better spatial distribution of start emissions, the start emission rates provided by EMFAC targets start emission to the location of the vehicle start. The assumptions for cold and hot start percentages are key to this approach. Any modifications to the start mode percentages would affect the emissions estimate. Use of local survey data to derive the start mode fractions, as was done in San Diego, would improve the emission estimate. Current emissions models, such as BURDEN and DTIM, could be used for the regional and gridded emissions inventories. This approach is a minor improvement to the current approach in California.

4.5.3 Operating Modes by Link

A more long-term approach to improving the emissions estimates from cold start is to develop a way to calculate emissions for each link by calculating an emission rate for the link based in the operating mode fractions for the link. This approach would utilize the network model output of volumes by operating mode and distribute the start emissions to the link where the emissions occurs. However, to compute

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emissions on a link-by-link basis is highly data intensive and the benefits of such an approach may not warrant this level of detail.

4.5.4 Further Research

During the process of preparing this chapter, several issues arose that would require further research. Some of these issues and potential approaches for research are identified below.

1. With all of the approaches recommended above, the need to compare the results to actual emissions estimates is important. While disaggregating travel activity and emission rates appear to be improvements to the current emissions estimation processes, the actual impact on the emissions estimate is not clear.

2. Since emission rates from EMFAC and MOBILE are not directly comparable, emissions inventory using emission rates from MOBILE5 should be compared to the results using EMFAC emission rates. The emission rates should be applied to the same travel activity data and the resulting regional emissions should be compared.

3. Similar to the study conducted using the TAPES model, operating mode fractions calculated using data from various regional network models should be compared. By summing the VMT by operating mode over all links in the networks, an overall mode fraction can be calculated and the resulting affects on the emission rates should be compared to that using the default FTP mode fractions. Modifications to these fractions should be evaluated. The effects that changes in the mode fractions have on the emissions rates should be assessed on a g/mile basis and the overall effect on the emissions inventory for a particular area should be determined.

4. Given the variability of operating mode fractions, additional study of the effects of changes to the FTP operating mode fractions should be conducted. A comparison of the g/mile emission rates should indicate how sensitive the rates are to the operating mode fractions. If possible, the results of this comparison should indicate when the disaggregation would have a significant effect on the composite emission rates produced by MOBILE5.

Chapter 5. Summary of Air Quality Analysis Post-Processing Methods

Each of the previous chapters discussed methods for improving the air quality analysis processes. The three methods discussed include improvements to the speed estimates produced from the model, post-assignment processes to improve speed estimates, and improvements to the prediction of trip by vehicle operating mode. This chapter summarizes the advantages and disadvantages of the methods recommended in the previous chapters.

5.1 Improved Speed Models

Improvements to the speed estimates from models can be divided into three components – estimating free-flow speed, estimating capacity, and the speed-flow relationship. Each was discussed in detail and evaluated in Chapter 2. The recommended methods are summarized here.

5.1.1 Free-flow Speed Estimation

For free-flow speed estimation, the recommendation was to use the NCHRP 3-55(2) free-flow speed equations recommended by Dowling (which take into account the posted speed limit, signal spacing, and signal timing), or the NCHRP 3-45 and HCM free-flow speed equations (which are sensitive to geometric design parameters and will be contained in the 1997 Highway Capacity Manual).

The Highway Capacity Manual and NCHRP 3-45 techniques allow the planner to estimate the speed reducing effects of geometric design factors and access point density. The HCM techniques may be used where the additional data is available to the planner. Planners, however, rarely have access to the necessary geometric design details such as lane width and lateral clearance. The HCM and NCHRP 3-45 techniques are also currently limited to a specific set of facility types. The NCHRP 3-55(2) method can be applied to any facility where the posted speed limit is known. However, the method is not reliable if local agencies have used "atypical" criteria for setting the speed limits.

5.1.2 Link Capacity Estimation

The recommended methods for estimating link capacities include the Florida LOS Manual general table of service volumes or the option of developing specific estimates of maximum service volumes using the Florida table generating spreadsheets. Those agencies with more resources available and desiring greater sensitivity to geometric conditions may choose to use the NCHRP 3-55(2) formulae for estimating capacity.

The Highway Capacity Manual is the most generally accepted basis for computing highway capacity. However, it requires data not frequently available to planners. Consequently, Florida DOT and others have used defaults for some of the needed data to make the HCM method more useful for transportation modeling and planning applications. The Florida LOS Manual provides one set of defaults for applying the HCM method. NCHRP 3-55(2) provides a procedure for applying the HCM that allows the selective substitution of defaults for those data items not available in a particular locality. Both the Florida and NCHRP 3-55(2) methods are poorly suited to estimating the capacity of arterials with no left turn bays and unprotected left turns. The HCM analytical process for this situation is difficult to approximate with a method suitable for planning purposes.

5.1.3 Speed-flow Relationships

Since the standard BPR curve underestimates mean vehicle speed for flows below capacity and overestimates speeds for demands greater than capacity and is insensitive to signal control parameters, it is recommended that planning agencies look into updating the parameters of their speed-flow curves to better reflect recent research on the impacts of volumes on freeway and arterial speeds. Upon review of several speed-flow relationships, one of the forms of the BPR curve recommended by Horowitz or Dowling was suggested since the change in parameters enhances its accuracy by making the BPR curve more consistent with the current HCM.

Horowitz also adds the capability to estimate node delay using procedures that approximate the 1985 HCM method. Although this improves consistency with the HCM, little is known about how the computation of node delay improves the accuracy of the speed estimates. Horowitz did find that the incorporation of node delay in the traffic assignment process results in the presence of multiple equilibria.

Alternatively, planning agencies may choose to use the Akcelik equation which has the advantage of being based upon queuing theory and may be computationally faster than the BPR curve. Akcelik proposes a speed-flow relationship based on time-dependent queuing with random arrivals. Limited testing by Dowling of Akcelik's equation suggests that this equation is superior to the standard BPR curve in its ability to replicate the speed estimates for "over capacity" conditions on freeways. The Akcelik equation however tended to over estimate delay for arterials at moderate volumes. The Akcelik equation is a strong candidate for replacing the standard BPR curve, however; given the established experience in the US with the BPR curve and its variations, the recommendation is to proceed with an update of the BPR curve for now, with future testing of the Akcelik equation perhaps justifying further refinements and its ultimate substitution for the admittedly heuristic BPR curve.

5.2 Assignment Post-Processors

Several assignment post-processing procedures are described in Chapter 3. Most of them attempt to apply the analytical methods contained in the Highway Capacity Manual to the estimation of capacity and speed in travel demand models. Since the Highway Capacity Manual, however, does not treat situations where demand exceeds capacity, many of the post-processor methods also include a method for computing the delays due to queuing. A few post processor methods completely avoid the HCM by fitting simple linear or curvilinear speed estimation equations to real world or simulated data.

Agencies considering the use of assignment post-processors need to weigh the increased data requirements of post processors against the desire for improved speed estimates. The addition of queuing analyses to the speed estimation process will enhance accuracy but runs the risk of over-estimating congestion by ignoring the impact of congestion on peak spreading. The major advantage of post processors is that they allow the planner to test the impacts of facility design and operation options that can not be tested in a traditional travel demand model. Post processors also extend the ability of planners to analyze traffic operations over the length of the peak period, rather than being limited to 24 hour or a peak hour analysis.

Some of the advantages and disadvantages of several post-processing procedures are briefly summarized in Table 29.

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Table 29. Summary of Assignment Post-Processors		
Method	Advantages	Disadvantages
Dowling and Skabardonis	Extends peak hour analysis to an entire peak period. Easily automated and applied to the entire model highway network.	Does not account for propagating effects of queue to upstream or downstream links
Margiotta NCHRP 7-13 (Lomax)	Predicts speeds based on ratio of daily traffic to hourly capacity. Predicts speeds as a function of signal density, access density, and v/c ratio	Linear techniques will be less robust than non-linear techniques. Unclear how well the linear equations would perform in queuing situations. Difficult to update them to changes in the HCM.
NCHRP 255 (Pedersen and Samdahl)	Complete method for applying the HCM method and queuing analysis to specific facilities on network. Easily extended to peak period analyses by repeating the analysis steps for each hour and carrying over excess demand.	Does not address multi-hour analyses. Not designed to deal with multiple queues that may interfere with each other.
HPMS Analytical Process	Accounts for pavement condition, grade, curves, stop cycles, idle time, and speed change cycles on selected facility.	Oriented to facility specific analyses. Difficult to update since it relies upon an extensive set of look-up tables and charts.
Ruiter	Extends the HCM method to over-capacity conditions by adding queuing delay equations.	Need to change the look-up tables for each new edition of the HCM. Need to solve the equations for many facility types with range of input variables.
Boston Central Artery	Extends the HCM to queuing situations. Automated speed estimation steps. Can be applied to the entire network.	Does not directly address multi-hour analyses.
DTIM2	Multi-hour queuing analysis procedure. Allows planners to specify default input data by facility type and area type. Automated and can be applied to the entire highway network.	May produce significantly lower speed estimates than traditional BPR curve based approaches. Currently investigating the addition of a peak spreading capability.
NCHRP 3-55(2)	Splits peak period into 1-hour time slices Splits the facility into series of segments. Queuing is identified and carried over to the following time slice. Accounts for signal control delays.	Oriented toward the analysis of a single facility. Queues are not propagated upstream, nor are downstream demands reduced due to upstream bottlenecks.

5.3 Prediction of Trips by Vehicle Operating Mode

Improvements to the emissions inventory modeling processes can be made to improve the ability to estimate emissions from vehicles in the start transient modes of operation. These improvements involve changes to operating mode fractions that are input to the MOBILE5 emission factor model as well as improvements and modifications to travel demand models to disaggregate the travel activity data.

With the variability of mode fractions by trip purpose, time-of-day, size of the urban area, facility type, and locations, one approach to improving the start emissions estimates is to modify the mode fractions used by MOBILE5. However, this approach does not provide an accurate spatial dispersion of emissions since it allocates the start emissions based on the VMT. The mode fractions do not provide the spatial-temporal distribution that becomes more important for CO emissions.

The travel demand models can be used to predict the operating mode fractions specific to that network coverage area. Software programs such as EMME2 and MINUTP allow the user to track trips that are made in the cold transient mode as they travel through the network. The model can provide link traffic volumes by operating mode. By summing the link VMT by operating mode, operating mode fractions of VMT can be calculated that are specific to the area being analyzed. These mode fractions can be input to MOBILE5 in place of the FTP defaults.

The travel models could be used to derive operating mode fractions by trip purpose, time-of-day, or other variables that have been shown to affect operating mode fractions.

This approach of alternative operating mode fractions can be taken even further by disaggregating mode fractions by more than one variable. Operating mode fractions can be cross-classified by trip purpose and time-of-day. The resulting emission rates would be specific to that particular trip purpose during the specified time period. The same approach could be applied by facility type, location or other link variable that can be used to disaggregate travel activity. The greater the disaggregation, the more of a bookkeeping challenge it will be to estimate the emissions.

Another approach is that used by CARB which separates start emissions from running emissions and provides a better spatial distribution of start emissions. To provide a better spatial distribution of start emissions, the start emission rates provided by EMFAC targets start emission to the location of the vehicle start. Use of local survey data to derive the start mode fractions would improve the emission estimate. Current emissions models, such as BURDEN and DTIM, could be used for the regional and gridded emissions inventories.

A more long-term approach to improving the emissions estimates from cold start is to develop a way to calculate emissions for each link by calculating an emission rate for the link based in the operating mode fractions for the link. This approach would utilize the network model output of volumes by operating mode and distribute the start emissions to the link where the emissions occurs. However, to compute emissions on a link-by-link basis is highly data intensive and the benefits of such an approach may not warrant this level of detail.

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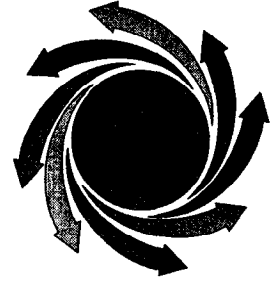
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